Laboratory Study of the Geotechnical Properties of Abraded Railway Ballast with Natural and Clay Mix Fouling

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LABORATORY STUDY OF THE GEOTECHNICAL PROPERTIES OF ABRATED RAILWAY BALLAST WITH NATURAL AND CLAY MIX FOULING

A Dissertation Presented

by

ANDREW K. ROHRMAN

Submitted to the Graduate School of the University of Massachusetts Amherst in partial fulfillment of the requirements for the degree of

DOCTOR OF PHILOSOPHY

May 2019

Civil and Environmental Engineering
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Approved as to style and content by:

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ACKNOWLEDGMENTS

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Finally I would like to thank all my family and friends who have supported me over all of these years. Your encouragement through this process is greatly appreciated.
ABSTRACT

LABORATORY STUDY OF THE GEOTECHNICAL PROPERTIES OF ABRADED RAILWAY BALLAST WITH NATURAL AND CLAY MIX FOULING

MAY 2019

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With 140,000 miles of active track, railroad freight is the largest method of transporting goods in the United States. An important component of the railroad substructure is the gravel-size aggregate placed below the tracks, called ballast. The ballast is responsible for draining water away from the track, resisting forces from traffic loading, maintaining track position, and reducing loads to acceptable pressures for the subgrade. Ballast is typically selected to be highly angular and uniformly graded, but these characteristics are constantly changing under dynamic traffic loading. As the ballast is loaded, the particles abrade against each other, grinding off stone dust and breaking off larger corners. This process reduces the angularity of ballast and also contributes to fouling, which is finer grained materials entering the voids of the ballast. Fouling generally reduces the strength of ballast, particularly when water is present, as it prevents the water from draining away from the track. This study aims to better understand the influence of both fouling content and ballast particle angularity on the strength properties and deformation characteristics of ballast. This is achieved through a series of laboratory tests using ballasts of different angularity and different fouling types, from which the results are directly compared to one another. Additionally, to better understand the particle packing structure of ballast-fouling mixtures, a series of minimum and maximum density tests have been performed on
several different mixes. The testing presented shows that angular ballast and abraded ballast achieve similar strengths, but the abraded ballast is much more susceptible to deformations, making it problematic in track. The introduction of clay fines into the fouling material shows significant loss of ballast strength, and also identifies issues with current methods of quantifying ballast. The density testing provides the first baseline for the range of possible densities of an angular and abraded ballast across a wide spectrum of fouling conditions. Finally, box testing of fouled abraded ballast has led to the first known box failure in a laboratory setting, revealing important characteristics of ballast.
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>ACKNOWLEDGMENTS</th>
<th>iv</th>
</tr>
</thead>
<tbody>
<tr>
<td>ABSTRACT</td>
<td>v</td>
</tr>
<tr>
<td>LIST OF TABLES</td>
<td>x</td>
</tr>
<tr>
<td>LIST OF FIGURES</td>
<td>xi</td>
</tr>
</tbody>
</table>

## CHAPTER

1 INTRODUCTION AND SCOPE .......................... 1

1.1 Introduction .................................. 1
1.2 Objectives and Scope of Research .......... 4

2 LITERATURE REVIEW ............................... 6

2.1 Introduction .................................. 6
2.2 Components of Railroad Track Systems .... 6

2.2.1 Rails ........................................ 8
2.2.2 Fastening System ............................ 9
2.2.3 Ties .......................................... 9
2.2.4 Ballast ...................................... 10
2.2.5 Subballast .................................. 12
2.2.6 Subgrade ................................... 12

2.3 Geotechnical Properties of Ballast ........ 12

2.3.1 Particle Size, Shape, and Roughness ...... 13
2.3.2 Gradation .................................... 14
2.3.3 Drainage ..................................... 17
2.3.4 Density ...................................... 18
2.3.5 Shear Strength ............................... 21
2.3.6 Friction Angle ............................... 23
2.3.7 Elastic Modulus .............................. 23
2.3.8 Deformation and Volumetric Strain ........ 24
2.3.9 Poisson's Ratio .............................. 26
2.3.10 Settlement .................................. 27
2.3.11 Resilient Modulus ......................... 29

2.4 Fouled Ballast .................................. 31
APPENDICES

A  ADDITIONAL TRIAXIAL TEST TABLES AND FIGURE .............. 95
B  BOX TEST SETTLEMENT AND SETTLEMENT RATE CURVES ........................................... 124

REFERENCES ......................................................... 132
**LIST OF TABLES**

<table>
<thead>
<tr>
<th>Table</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10</td>
</tr>
<tr>
<td>2</td>
<td>16</td>
</tr>
<tr>
<td>3</td>
<td>48</td>
</tr>
<tr>
<td>4</td>
<td>51</td>
</tr>
<tr>
<td>5</td>
<td>53</td>
</tr>
<tr>
<td>6</td>
<td>67</td>
</tr>
<tr>
<td>7</td>
<td>72</td>
</tr>
<tr>
<td>8</td>
<td>96</td>
</tr>
</tbody>
</table>

1. Typical Tie Dimensions [58]  
2. AREMA recommended gradations for new ballast [3]  
3. Mill Abrasion test results for granite and abraded ballasts.  
4. Percentage of minerals determined by XRF tests on ballast and fouling samples.  
5. Values of % fouling and corresponding PVC values for each fouling material.  
6. 50% secant modulus for all triaxial tests.  
7. Loading schedule for box tests.  
8. Friction angles and ultimate strengths for all triaxial tests.
<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Components of the track structure [58]</td>
<td>7</td>
</tr>
<tr>
<td>2</td>
<td>Influence of angularity on the friction angle of aggregates [66]</td>
<td>14</td>
</tr>
<tr>
<td>3</td>
<td>Example grain size distribution curves for clean and fouled ballast [58]</td>
<td>15</td>
</tr>
<tr>
<td>4</td>
<td>Definitions of different types of grain size distributions [58]</td>
<td>16</td>
</tr>
<tr>
<td>5</td>
<td>Sources of water infiltrating into the ballast [58]</td>
<td>17</td>
</tr>
<tr>
<td>6</td>
<td>Results of model comparing water tables in clean ballast and moderately</td>
<td>19</td>
</tr>
<tr>
<td></td>
<td>fouled ballast after a rain event [55].</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>Linear relationship between relative density and the difference in peak and</td>
<td>21</td>
</tr>
<tr>
<td></td>
<td>residual strength of clayey-sand fouled ballast [13]</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>Normalized Shear Strength to Normal Stress Relationship for tested basalt</td>
<td>22</td>
</tr>
<tr>
<td></td>
<td>ballasts [35]</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>Relationship of friction angle and confining pressures and the role of</td>
<td>24</td>
</tr>
<tr>
<td></td>
<td>particle breakage [37]</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>Influence of cell pressure on Poisson’s Ratio and initial deformation or</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>elastic modulus [35]</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>Typical volumetric strain and deviator stress curves for ballast [35]</td>
<td>26</td>
</tr>
<tr>
<td>12</td>
<td>Plot showing settlement of ballast in 1-D compression test under different</td>
<td>28</td>
</tr>
<tr>
<td></td>
<td>loading conditions [32]</td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>Relationship between resilient modulus and effective confining pressure for</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>different amounts of particle breakage [34]</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>Relationship between resilient modulus and bulk stress [34]</td>
<td>30</td>
</tr>
<tr>
<td>15</td>
<td>Sources of Fouling [58]</td>
<td>31</td>
</tr>
<tr>
<td>16</td>
<td>Schematic diagram of (a) clean, (b) partially fouled, and (c) totally</td>
<td>32</td>
</tr>
<tr>
<td></td>
<td>fouled ballast [36]</td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>Elastic modulus as a function of percentage fouling [7]</td>
<td>37</td>
</tr>
<tr>
<td>18</td>
<td>Characteristics of Packing Density due to fines content for steel shots</td>
<td>39</td>
</tr>
<tr>
<td></td>
<td>[11]</td>
<td></td>
</tr>
<tr>
<td>19</td>
<td>Characteristics of void ratio versus fines content for sand-silt mixtures</td>
<td>40</td>
</tr>
<tr>
<td></td>
<td>[11]</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>View of spent ballast on site in Shelburne Falls, MA.</td>
<td>44</td>
</tr>
<tr>
<td>21</td>
<td>Example of some naturally abraded ballast pieces.</td>
<td>45</td>
</tr>
<tr>
<td>22</td>
<td>Grain size distribution curves for the Shelburne Falls, MA abraded ballast</td>
<td>46</td>
</tr>
<tr>
<td></td>
<td>with AREMA #4 gradation limits.</td>
<td></td>
</tr>
<tr>
<td>23</td>
<td>Sieve analysis results for natural fouling material.</td>
<td>47</td>
</tr>
<tr>
<td>24</td>
<td>Atterberg limits results for several natural fouling and Prestige clay</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td>mixtures.</td>
<td></td>
</tr>
<tr>
<td>25</td>
<td>Triaxial test setup in load frame. Instron control panel is shown to the</td>
<td>55</td>
</tr>
<tr>
<td></td>
<td>left; the cell pressure control panel is shown to the right.</td>
<td></td>
</tr>
<tr>
<td>26</td>
<td>Typical stress-strain curves.</td>
<td>57</td>
</tr>
<tr>
<td>27</td>
<td>Stress-strain curves for the abraded ballast with natural and Prestige</td>
<td>58</td>
</tr>
<tr>
<td></td>
<td>mix fouling at different water contents.</td>
<td></td>
</tr>
</tbody>
</table>
CHAPTER 1
INTRODUCTION AND SCOPE OF RESEARCH

1.1 Introduction

According to the Federal Railroad Administration, the United States moves more freight by rail than any other freight rail system worldwide. With 140,000 miles of track, approximately 40\% of US freight is carried by rail, making it an important component of the economy. Increased demand has resulted in higher speeds, axle loads, and traffic density. The infrastructure supporting these demands is therefore of great importance. Railroad track in the US is most commonly supported by ballast, which is a uniformly graded, gravel-size aggregate. The ballast serves primarily to drain water away from the track, but it is also essential in resisting forces in the vertical, lateral, and longitudinal directions, maintaining track position, and distributing loads to acceptable pressures for the subgrade [58]. Typically ballast is selected to have high angularity, shear strength, and hardness in order to sufficiently serve these functions [37].

Ballast is the primary material used for the track substructure in the United States, largely because of the availability of suitable rock in most areas, as well as its low cost and simple maintenance methods [37]. Other methods, such as slab track, are becoming more popular in Europe and Asia, but are still not common in North America. While ballast has many advantages, there are some issues with the material that require special maintenance and monitoring. The primary issue of concern is the introduction of fouling, or finer aggregates entering the void spaces of the ballast. Commonly, fouling is defined as particles passing the 3/8” (9.5-mm) sieve [58]. The most common source of fouling is breakdown fouling [42], in which the ballast particles have impact under loading, breaking off corners or grinding together to form stone dust. Fouling can also infiltrate the track from the subgrade below or be transported
into the ballast by wind or water [58]. The introduction of fouling into the ballast has negative impacts on its performance, particularly when water is also present in the track. Under loading, the ballast properties are constantly changing, making parameters such as fouling content, fouling type, and ballast angularity of interest to researchers. This study aims to investigate these parameters through laboratory testing.

The effects of fouling on the geotechnical properties of ballast have been investigated in many studies over the past several decades. Laboratory testing has shown that fouling has negative impacts on ballast and track performance by increasing the rate of deterioration, reducing track resiliency, and reducing the ability of ballast to withstand the dynamic loading forces from traffic. [18] [58] [68]. While it has been shown that fouling can increase ballast strength when dry [40], the resulting poor drainage creates many negative effects when wet. Increased moisture from fouling can decrease the shear strength of ballast, reduce track stiffness, and decrease track stability. [30] [33] [64]. Most of these studies focus on quantity of fouling, rather than type of fouling. There are limited cases in which materials apart from breakdown fouling are used. Some studies have used natural fouling materials taken from track [16] [64]. There is one study in which pure clay fouling was used [36]. This study found that the clay decreased dilation, but any benefit gained from this was negated by the loss of peak and post-peak strength. Additionally, large amounts of clay reduced compression, while smaller amounts acted as a lubricant between ballast particles, increasing compression. None of the studies regarding ballast fouling had any direct comparison of results from different types of fouling materials. The research presented in this dissertation will compare three different fouling types in comparable tests so that better conclusions can be drawn about how they affect ballast performance.

As mentioned previously, the breakdown of ballast is the most common source of fouling, but a consequence of this is that the ballast particle shape is changing over the
course of its life. Mechanisms such as attrition, grinding, and impact all contribute to the loss of ballast angularity [48]. Some past work has shown that friction angles of granular materials increase with increasing angularity [44] [28]. A study by Huang [29] showed that increased angularity results in improved track stability and strength properties due to better particle interlock. There are some studies, however, that show minimal correlation between ballast angularity and strength [27, 66]. It is also important to note that increased ballast angularity can result in looser packing, creating larger void spaces which allows that ballast to accommodate higher quantities of fouling. The importance of angularity on these facts it makes it of continued interest to researchers.

The solids density of the parent rock is of importance when selecting ballast materials, but the bulk density of the aggregate should also be considered. It has been shown that ballast should be placed at the maximum possible density [54]. However, the American Railway Engineering and Maintenance-of-Way Association does not specify density or compaction criteria in its Manual for Railway Engineering. It does indicate that after distribution of ballast, tamping of ties should be carried out by means of mechanical tampers. It is clear that achievement of high density is important, though it is difficult to clearly define what that is. This dissertation outlines a series of tests which were completed to determine the maximum and minimum densities of two different types of ballast. Additionally, these tests are performed on ballast in several different fouling conditions to better assess ballast density during. This data will be used to eventually develop a mixing model based on the work by Chang et al. [12] for silt-sand mixtures. The initial minimum and maximum density results show that a ballast-fouling mixing model will work in a similar manner.

This research program focuses on a series of laboratory tests which builds upon previous work by Kashani [40]. Laboratory testing, particularly triaxial tests, have been extensively used to study ballast [6, 23, 35, 36, 40, 43, 50, 63, 64]. This study
hopes to provide new insight by performing a large number of tests which can directly compare the variables of fouling content, fouling type, and ballast angularity. A series of triaxial tests and modified box tests are prepared for this analysis. Additional work was also performed to analyze material properties of the ballast and fouling samples used. This testing resulted in the first known failure of ballast in a box test setting, providing valuable insight to ballast behavior at the point of failure.

1.2 Objectives and Scope of Research

The main objective of this study is to gain a better understanding of the behavior of ballast at the point of failure. Within this there are several different parameters of interest which effect behavior. As a result, there are also objectives to determine how properties such as particle angularity, fouling materials, fouling contents, and moisture conditions effect the behavior of ballast. Another objective is to evaluate the relative density of ballast through minimum and maximum density testing, which has not previously been investigated.

The intent of this research is for others to be able to use the acquired data to predict ballast behavior and potential track failures based on the condition of fouled ballast. The scope of this research is as follows:

- Identify candidate abraded ballast material and two fouling materials to be used for testing

- Determine basic material properties through laboratory testing such as grain size distribution, Mill Abrasion, and Atterberg Limits

- Conduct static triaxial CIDC tests at three fouling conditions, three moisture conditions, and three confining pressures for each material. A total of 45 triaxial tests are included in this study
• Run dynamic box tests on ballast in different fouling conditions using equivalent Heavy Axle Loads (HAL) to determine dynamic behavior of the ballast

• Compare the results of all tests to those previously conducted by Kashani [40] on highly angular ballast

• Conduct minimum and maximum density tests on different ballast types at different fouling conditions and apply results to the mixing model developed by Chang et al. [12]
CHAPTER 2
LITERATURE REVIEW

2.1 Introduction

The following sections contain a summary of the literature review completed in anticipation of this research. Each section serves to provide context for the work done and to provide an understanding of the need for such work. Included is some background on the components of railroad track systems, the geotechnical properties of ballast, and the importance of fouling and its effects on ballast performance. Each of these sections include supporting data from previous studies to better understand the work that has been done thus far to properly characterize the behavior of fouled ballast. This should indicate where knowledge is lacking or insufficient, providing rationale for the dissertation research.

2.2 Components of Railroad Track Systems

The track system is broken down into two major parts; the superstructure and the substructure. The track superstructure consists of the rail, fastening system and ties. The substructure consists of the granular material installed for the purposes of drainage, anchorage of the track superstructure, distribution and transfer of loads, facilitation of track adjustment and alignment, and shielding of the roadbed from climatic forces. These materials include the ballast, subballast, and subgrade or roadbed. [3]

Figure 1 illustrates the different track components as well as the axes that are used to describe the orientation of these components. Each component will be described in greater detail in the following sections.
Figure 1: Components of the track structure [58]
2.2.1 Rails

The components of the track system on which the train cars sit are the rails. These are steel members that run longitudinally to the track and serve to transfer the concentrated wheel loads to the ties below. Because the ties are spaced apart and do not provide continuous support, the rails must have considerable stiffness so that they do not deflect over the unsupported spans. Additionally, the shape of the rail (as well as the wheels) will dictate how the trains are guided on the track. The American Railway Engineering and Maintenance-of-Way Association (AREMA) recommends using common rail sections of 115, 136, and 141 lb/yd (57.0, 67.5, and 69.9 kg/m) to meet the requirements of the rails’ function [3]. The distance between rails is known as the gage, and varies globally. The gage of rails in North America is typically 56.5 inches (1435 mm). [58]

A continuous smooth rail surface is desirable to reduce train vibration, improving the ride and track performance. However, segments of the rail must be connected, usually by bolted joints or welding. Rail joints should connect rails in such a way that they act as a continuous girder, and should match the deflection resistance of the rail itself as closely as possible. The joints must also prevent relative movement of two rail sections in the lateral and vertical directions, but should allow movement in the longitudinal direction to prevent rail buckling or breakage induced by expansion and contraction of the rails. [3]

Discontinuity in rail surface can cause impact loads and reduced rail stiffness. The resulting higher stresses on the ballast and subgrade layers can lead to higher settlements at the joints. Additionally, track deterioration is higher at these locations due to increased tie wear and ballast fouling. Continuously welded rail eliminates joints and therefore removes the above described issues that arise from joints. However, it has a higher chance of buckling, it is more difficult to replace, and it is more expensive to install and transport. [58]
2.2.2 Fastening System

The main purpose of the fastening system is to fasten the rail to the ties, but its functions can vary depending on the type of tie used in the track. Generally, fasteners provide gage restraint, resist forces and transfer them from rail to tie, and distribute bearing forces. Other purposes include electrical isolation for charged rail and conservation of seat cant (slope between the two rails) on curved sections of track. When wooden ties are used, steel tie plates of sufficient size to distribute loads to an acceptable bearing pressure for wood are required. The plates also prevent mechanical wear on the ties and assists the fastening system in resisting lateral forces and maintaining cant. Concrete ties do not provide the resiliency afforded by wood ties, so resilient pads must be used. Besides increasing resiliency, these pads also damp wheel vibrations, reduce contact attrition between the rail and tie, and provide electrical insulation. [58] [3]

2.2.3 Ties

The ties (also referred to as crossties or sleepers, depending on region of the world), are the elements that lie below the rails, perpendicular to the direction of the track. The ties are responsible for distributing rail loads to the supporting ballast layer, providing anchorage points for the rail fastening system which maintains track gage, and to prevent rail movement in the lateral, longitudinal, and vertical directions by securing the superstructure to the ballast. [58]

Ties were initially made almost exclusively of timber because it was a readily available natural resource. However, wood ties are susceptible to rot, insect infestation, and other environmental effects. Subsequently, steel was commonly used for tie production for over 50 years in certain parts of the world, including Europe. Over time, increased train loads and speeds required heavier ties, which led to the use of concrete ties. Two types of concrete ties were developed. The first is twin-block ties,
which use two reinforced concrete blocks under the rails, connected by a steel bar. The second is monoblock ties, which use a single rigid beam of pre-stressed concrete. Damaged ties can be replaced in maintenance operations. Typically, concrete is used for such replacements, though timber is also used, particularly in locations where sub-grade quality is poor. Steel ties are no longer used when replacements are necessary. Worldwide, concrete ties make up 20% of the ties used in tracks. [53]

Concrete ties have the advantage of providing better fastener security than wood ties and are potentially more durable. This means that they provide better rail restraint overall. However, wood is easier to handle since it is lighter, and concrete ties do not provide as much resiliency, so pads are required. Tie dimensions vary around the world, and common dimensions can be seen in Table 1.

Table 1: Typical Tie Dimensions [58]

<table>
<thead>
<tr>
<th>Location</th>
<th>Material</th>
<th>Width (mm)</th>
<th>Length (mm)</th>
<th>Spacing (mm)</th>
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<tbody>
<tr>
<td>Australia</td>
<td>Wood</td>
<td>210-260</td>
<td>2000-2743</td>
<td>610-760</td>
</tr>
<tr>
<td></td>
<td>Concrete</td>
<td></td>
<td></td>
<td>600-685</td>
</tr>
<tr>
<td>China</td>
<td>Wood</td>
<td>190-220</td>
<td>2500</td>
<td>543-568</td>
</tr>
<tr>
<td></td>
<td>Concrete</td>
<td>240-290</td>
<td>2500</td>
<td>568</td>
</tr>
<tr>
<td>Europe</td>
<td>Wood</td>
<td>250</td>
<td>2600</td>
<td>630-700</td>
</tr>
<tr>
<td></td>
<td>Concrete</td>
<td>250-300</td>
<td>2300-2600</td>
<td>692</td>
</tr>
<tr>
<td>North America</td>
<td>Wood</td>
<td>229</td>
<td>2590</td>
<td>495</td>
</tr>
<tr>
<td></td>
<td>Concrete</td>
<td>286</td>
<td>2629</td>
<td>610</td>
</tr>
<tr>
<td>South Africa</td>
<td>Wood</td>
<td>250</td>
<td>2100</td>
<td>700</td>
</tr>
<tr>
<td></td>
<td>Concrete</td>
<td>203-254</td>
<td>2057</td>
<td>700</td>
</tr>
<tr>
<td></td>
<td></td>
<td>230-300</td>
<td>2200</td>
<td>600</td>
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</tbody>
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2.2.4 Ballast

It is often said in industry that the primary purpose of ballast is to facilitate the drainage of water away from the track. But this is just one of many important functions that the ballast serves. The ballast acts as a structural element, supporting and distributing loads to reduce the bearing pressure on the subgrade. The ballast also serves as an anchor for the ties which resists vertical, lateral, and longitudinal forces applied. This maintains the position of the track while increasing its resiliency and energy absorption. As part of its function facilitating drainage, ballast also
provides large voids, which gives space to store fouling and allow particle movement through the ballast. [58]

Ballast consists of a selected crushed stone. Ideal ballast has high angularity, high shear strength, high toughness and hardness, resistance to temperature changes and chemical attack, low absorption, and is free of cementing properties [37] [3]. It is important to note that many of the functions of ballast require contradictory characteristics to fulfill them. For instance, free drainage requires a generally uniform gradation with large voids in the ballast, but higher shear strengths can be achieved with a more broadly graded ballast. There is no single ballast that can equally satisfy all the functions required, so ballast must be selected based on the needs of the particular track being designed [52]. Some of the commonly used rock types which generally meet these criteria are granite, traprock, quartzite, carbonate rocks, and slags [3].

For successful ballast performance, the most important properties for the individual rock include petrology, durability, shape, and surface roughness. The important properties of the bulk material include permeability, bulk density and specific gravity [37]. The bulk properties are also referred to as the physical state, and is often directly influenced by the tamping or mechanical compaction that is used at the initial placement of ballast or during maintenance [58].

Ballast continues to be widely used because construction is generally low in cost, and it lends itself easily to maintenance when required [37]. While ballasted track is the most common rail structure, in some countries, such as Germany and Japan, ballast and ties are being replaced by slab track, which usually consists of a continuous concrete footing which supports the rails [53].
2.2.5 Subballast

The subballast is typically a broadly graded mixture, often consisting of crushed stone, crushed gravels, natural or manufactured sands, crushed slag, or a mixture of these. The gradations commonly used for subballast are often similar to those used in highway subgrade specifications [3]. Subballast serves to aid the ballast in reducing pressures from the ties to the subgrade. Perhaps more importantly, it acts as a filter between the ballast and subgrade. This function prevents infiltration of the subgrade into the ballast by inter-penetration, upward migration, and attrition. Ultimately, this prevents degradation of the ballast by fouling. Track without subballast typically requires more maintenance because of increased fouling. [58]

2.2.6 Subgrade

The subgrade provides a stable foundation to the other layers of the substructure. The subgrade has a large influence over track performance, in part because traffic stresses can extend up to five meters below the track. The majority of this depth consists of the subgrade, which ultimately influences superstructure support resiliency, elastic deflection of the rail, and differential settlements of the track. Subgrade usually consists of in-situ soils, though fills are sometimes used on sloped or rugged terrain. Since subgrade material typically cannot be selected, the ballast and subballast are usually chosen to protect the subgrade so that it can continue to perform over time. [58]

2.3 Geotechnical Properties of Ballast

The properties of ballast, whether pertaining to the individual particle or the bulk material, play important roles in the performance of the track. These properties can be difficult to understand, as they are progressively changing over the life of the ballast as dynamic loading leads to particle breakage, material deformation, and
fouling [37]. Laboratory testing has been performed to better characterize the effects of these properties on ballast performance under many different conditions. These properties are outlined and discussed in the following sections.

2.3.1 Particle Size, Shape, and Roughness

Many early studies on particle shape and size produced varied results, making it difficult to fully understand their effects on the strength characteristics of granular materials. This variation was due in part to the inability to isolate variables within the testing schemes. A study by Vallerga et al. in 1957 [66] solved these issues, and found that particle size did not greatly affect the angle of friction of uniformly graded materials up to 0.2 inches (0.508 cm).

Particle shape can be quantified using sphericity, angularity or roundness. The angularity describes the sharpness of the corners on an individual particle. Sphericity describes how much a particle shape deviates from that of a perfect sphere, and can be measured using the equation

$$S_p = \left( \frac{6V}{\pi} \right)^{\frac{1}{3}}$$

where $V$ is the volume of the particle and $L$ is the diameter of the smallest sphere that would circumscribe the particle. Quantifications such as sphericity and roundness are often estimated visually, as exact measurements require use of a projected image.

Ballast is commonly selected to be highly angular, which has been argued to increase track stability and improve high-load bearing characteristics of the track [37]. However, Vallerga et al. [66] found that an increase in angularity of particles only produces a slight increase in angle of internal friction, as shown in Figure 2. This is supported by findings in an earlier study by Holtz & Gibbs [27]. By contrast, studies by Indraratna and colleagues have shown that angular rock has higher strength than rounded river rock [35] [37]. These results are supported by other work which
has shown that increased angularity results in increased friction angles of granular materials [44] [28]. More recent work by Huang used DEM modeling to show that increased angularity provides more interlock between ballast particles, leading to increased strength and track stability [29]. There are studies which both support and oppose the idea of increased angularity leading to significant improvements in ballast strength, making it of continued interest to researchers. It should also be noted that angularity may have positive effects on ballast void ratio. It has been shown that higher angularity ballast can have a looser configuration, creating more void space which can accommodate larger quantities of fouling [22].

![Figure 2: Influence of angularity on the friction angle of aggregates [66]](image)

The surface roughness of particles also play an important role in the strength characteristics. In fact, it has been shown that granular materials of uniform gradation can be greatly affected by surface roughness, with variations in friction angle as large as 8 degrees [66]. Again, this study was not performed on ballast, but provides insight to how surface roughness may influence ballast behavior.

### 2.3.2 Gradation

The gradation of the ballast has direct effect on the shear strength, drainage, and overall stability. Gradation is determined by sieving and washing material in general
accordance with ASTM D442 [5]. The mass of material captured on each sieve is taken, and results are typically presented in a plot of percent of material passing each sieve versus the grain size on a logarithmic scale. An example, given in Figure 3, shows a typical gradation for new, clean ballast, and another for a heavily fouled ballast.

![Figure 3: Example grain size distribution curves for clean and fouled ballast [58]](image)

New ballast is typically uniformly-graded meaning it has a narrow range of grain sizes. When ballast becomes fouled, it usually has two distinct grain sizes, which is termed gap-graded. Soils can also be called broadly-graded meaning there is a wide range of grain sizes, but this is usually not seen with ballast. Visual representation of these kinds of grain size distributions are presented in Figure 4. AREMA has standard size designations based on recommended ballast gradations which are presented in Table 2.

Triaxial testing of granular materials have had different results when interpreting shear strength of a material based on gradation. Some studies, such as the one conducted by Marachi et al. [47] show that strength increases with decreased particle size. Other studies show an increase in strength with increasing particle size [15]. It was earlier discussed that particle size has little effect on shear strength as shown
in the study by Vallerga et al. [66]. This notion has been expanded upon with the parallel gradation technique, which Varadarajan et al. [67] noted was a suitable technique for reducing the size of ballast to determine geotechnical properties through modeling. Further studies seem to confirm this, showing that parallel gradations of materials produce similar behaviors in some cases [8]. This could indicate that the relative sizes of particles within a specimen is more important than the minimum and maximum particle size of a specimen.
2.3.3 Drainage

When track is exposed to water, it can enter the substructure from precipitation falling directly on the track, surface flow from higher ground, and upward seepage from the subgrade, as shown in the illustration in Figure 5. Track drainage is very important to the performance of the track as many problems can arise from excess water in the substructure. Pore pressures can increase under cyclic loading conditions, which leads to increased plastic strains and decreased stiffness and strength. Other issues include subgrade attrition, hydraulic pumping, volume change from swelling, frost heave, and slurry abrasion which can cause ballast degradation and tie attrition. These issues increase the amount of maintenance required by the track, highlighting the importance of adequate drainage from the ballast. [58]

![Figure 5: Sources of water infiltrating into the ballast](image)

Typical hydraulic conductivity values were determined for ballast in various conditions of fouling by Parsons in 1990 [51]. This study found that clean ballast has a hydraulic conductivity of 1-2 inches/sec (2.54-5.08 cm/sec), while it is reduced to less than 0.0002 inches/sec (0.0005 cm/sec) for highly fouled ballast. Full scale laboratory testing by Heyns [24] showed that water entering the ballast due to rainfall sheds quickly, but the water table rises within the subballast and slowly falls after the rainfall stops. These tests showed that when the subgrade is horizontal, the water
table falls much more slowly than when it is sloped.

Rushton and Ghataora [55] created a numerical model based on Heyns experimental results using a Dupuit-Forchheimer approximation. One of the uses of this model was to investigate reduced permeability of the ballast due to fouling. Selig and Waters [58] stated that the permeability of moderately fouled ballast is about 10% that of clean ballast. The model used this information to investigate the water table in the ballast and subballast layers in these two conditions. The results showed that after 12 hours, the water table in the clean ballast dropped significantly, while the water table in the moderately fouled ballast was at 45% of the maximum after 12 hours. These results can be seen in Figure 6. This clearly shows that the presence of fouling has a large impact on the ability of the ballast to perform its function of draining water away from the track.

2.3.4 Density

The bulk density of ballast plays an important role in the stability of the track. It is a function of the specific gravity of the particles, as well as the void ratio of the ballast. Generally, a higher bulk density is desirable in ballast. This can be achieved by either increasing the particle density, or by making the material more broadly graded. Since a broadly graded ballast is not desirable, the particle density can be increased by selecting parent rock with a higher specific gravity. It has been shown that higher specific gravity in ballast can increase the tracks strength and stability, increase the cyclic durability, and reduce settlement. Additionally, increased specific gravity can lower ballast degradation and improve lateral stability on curved sections of track. These improvements show that bulk density is extremely important when it comes to track safety. [37]

The initial density of the ballast effects the other properties of the ballast. It has been determined that ballast should be placed at the highest density that can be
achieved [54]. This is one of the reasons why compaction of ballast is so important. Compaction is often achieved by tamping and surface compaction of the ballast when it is freshly placed or when maintenance is performed [35]. It should be noted that when maintenance is done, the track should be disturbed as little as possible. This is because the application of cyclic loading from train traffic increases the shear strength of the ballast, due in part to the densification of the material [63]. Tamping disturbs the material structure reducing strength. This is why the method of stoneblowing,
which removes fouling without disturbing the ballast, is preferable, as it maintains the more dense and compact structure of ballast after many cycles of loading.

Another important determinant of ballast behavior is the relative density. The relative density describes how dense or loose a granular material is in relation to the maximum and minimum possible density conditions. It is described using void ratios in the following equation:

\[
D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100\%
\]  

(2)

This equation can also be expressed using densities and unit weights. The maximum and minimum densities of granular materials can be determined using ASTM D4253 and D4254. [26] However, this methodology has not been used on ballast, instead relying on methods such as Proctor compaction to determine maximum densities.

A recent study by Chen and Zhang [13] investigated the effects of relative density on clayey-sand fouled ballast. It was found that an increase in relative density lead to increased shear strength and friction angle. Similarly, increased relative density also increased specimen dilatancy, particularly at low confining pressures. It was also shown that increased relative density increased the difference between the peak and residual friction angles for each test. However, this difference is reduced as the confining pressure increases for the same relative density. This relationship, shown in Figure 7, indicates a linear trend. It is clear that together, relative density and confining pressure have a significant impact on the behavior of fouled ballast.

The effects of fouling quantity on the relative density of ballast-fouling mixtures has not been researched. This research program fills some of the need in this area, and will be coupled with an existing sand-silt mixtures model, which is discussed further in the Modeling section.
2.3.5 Shear Strength

High shear strengths are necessary in ballast to support the high axle loads from rail cars. Shear strengths are a function of the normal stresses exerted on a soil. In the case of ballast, axial forces are related to the train loads, which have static values ranging from 12,000 lb (53.38 kN) for light passenger rail to 39,000 lb (173.5 kN) for heavy freight trains [58]. Effective lateral stresses are provided by the self-weight of the ballast shoulders (which is influenced by the degree of initial compaction) and transient stresses developed between the ties. This typically amounts to no more than 140 kPa (20.31 psi) [35]. Triaxial testing can be used in the lab to determine shear strengths by using analogous conditions. Specimens can be prepared with gradations typically used for railways, and compacted to densities commonly found in the field. Relatively low confining stress representative of the lateral stresses discussed previously can be applied to get meaningful results.

Indraratna et al. [35] developed a normalized relationship which serves as a criterion that describes the behavior of ballast at the low confining pressures typically
\[ \frac{\tau'_f}{\sigma_c} = m \left( \frac{\sigma'_n}{\sigma_c} \right)^n \]  

where \( \tau'_f \) is the shear strength, \( \sigma_c \) is the uniaxial compressive strength of the parent rock, \( \sigma'_n \) is the effective normal stress, and \( m \) and \( n \) are dimensionless constants. It was found that at low confining pressures, normalized shear strengths plotted against the ratio of normal stress to parent rock compressive strength on a logarithmic scale, all results fall within a narrow band, as seen in Figure 8. These results, which compare basalt ballasts only, are independent of compressive strength, void ratio, and particle size distribution.

Figure 8: Normalized Shear Strength to Normal Stress Relationship for tested basalt ballasts [35]

When discussing the shear strength of ballast, it is also important to consider cyclic effects. An investigation conducted by Suiker et al. [63] compared the shear strength of virgin ballast under static loading with that of ballast that had previously undergone 1,000,000 loading cycles using triaxial tests. The study found that ballast strength increased by 9% after undergoing cyclic loading. This is a smaller increase than what was observed in subballast material, which had a strength increase of
The smaller increase for ballast is likely influenced by its lower susceptibility to compaction brought on by its uniform gradation. Clearly, railway substructure materials under repeated service loads increase in strength, with a more significant effect when the materials are more susceptible to compaction. While no tests were run on fouled ballast, it is likely that a similar increase in strength would occur, the degree of which would likely fall somewhere between that of the tested ballast and subballast.

2.3.6 Friction Angle

The angle of internal friction is an important parameter for characterizing the strength of ballast. This property of ballast is commonly determined by means of large scale triaxial tests. The friction angle is directly influenced by the stress condition of the material. The relationship of friction angle and effective confining pressure can be seen in Figure 9. It has been found that at large normal stresses, the friction angle decreases, in part because of particle crushing. Ballast more commonly experiences low lateral pressures, which leads to higher friction angles. This is believed to be due to the inter-particle contact forces being well below the crushing strength of the parent rock, providing higher strength. Additionally, the increased ability of interlocking particles to dilate under low confinement may contribute to this [35].

Water content also has an impact on the friction angle. Duong et al. [16] found that friction angles decrease with increasing water content, and cited similar results from Seif El Dine et al. [57], Indraratna et al. [33]; Selig and Waters [58], Fortunato et al. [20] and Ebrahimi [17].

2.3.7 Elastic Modulus

The elastic modulus describes the stress-strain response in the elastic deformation zone. It is a measure of recoverable deformation behavior. For rail, it is an important
component in understanding the vertical stiffness of the track. Commonly, the elastic response can be characterized by the initial tangent modulus, taken from a stress-strain curve. Stress-strain curves are influenced by confining pressures, and thus the elastic modulus is as well. A study from Indraratana et al. [35] showed that the elastic modulus increases with up to 180 kPa (26.11 psi) of confining pressure. Beyond this point, the increases in elastic modulus became insignificant. However, confining pressures only reach 250 kPa (36.26 psi) in this study, which is typical for ballast since the thickness often only reaches around 1.5 meters (60 inches). A plot of the elastic modulus results is given in Figure 10. Poor performing ballast can have an elastic modulus as low as 13.7 MPa (1987 psi) while values up to 27.5 MPa (3989 psi) can be achieved by good performing ballast [49].

2.3.8 Deformation and Volumetric Strain

Granular materials will accumulate plastic, unrecoverable, deformations from repeated loading. It is therefore important to understand these deformations with respect to the behavior of ballast. Shear strains accumulate when the vertical stresses
are greater than the horizontal stresses. This condition occurs when a train loads the track, which leads to lateral spreading and vertical shortening of the material as the particles either rearrange into a denser packing or break into smaller particles which fill the voids of larger particles. [58]

Many studies have been conducted in the laboratory using both static and cyclic loading tests which have helped develop a better understanding of deformation behavior of ballast [35] [63] [16]. Using the low confining pressures typically seen in ballast, specimens commonly contract under load before dilating. Figure 11 shows typical stress-strain and volumetric strain response of ballast under monotonic loading.

Laboratory testing by Suiker et al. [63] further investigated the volumetric strains and deformation behavior of ballast during a study utilizing triaxial tests. It was shown that for the first 100 to 1000 loading cycles, the ballast undergoes what is known as conditioning, in which elastic volumetric deformations decrease and the material becomes stiffer. The cycles following the conditioning phase show elastic strains that are much larger than the plastic strains, and the elastic strains remain approximately constant, indicating material compaction. Additionally, measurement of permanent deformations decreased as the number of cycles increased. After a con-
Figure 11: Typical volumetric strain and deviator stress curves for ballast [35]

A considerable number of cycles, the ballast will enter a phenomenon known as shakedown, in which the cyclic response of the ballast will become almost purely elastic.

2.3.9 Poisson’s Ratio

One indicator of deformation is Poisson’s Ratio, which is given as the ratio of horizontal strain to vertical strain:

\[ \nu = \frac{\varepsilon_h}{\varepsilon_v} \]

Poisson’s Ratio captures both vertical and horizontal strains and can therefore give a glimpse of the overall deformation behavior of a material. Values typically fall between 0 and 0.5 for most materials, indicating that the volumetric strain is contractive under compression and less than axial strain. Poisson’s Ratio of 0 indicates that the volumetric strain is equal to the axial strain, while a value of 0.5 is given if no volumetric strains occur under load. Poisson’s Ratio greater than 0.5 is achieved if the material dilates under compressive axial strain. This is impossible for isotropic linear elastic materials, but can occur in ballast and dense granular soils. A typical
in-situ Poisson's ratio of 0.4 can be assumed for railroad ballast [58].

When considering the Poisson's ratio of soils, the confining stresses on the material play an important role in the deformations, and must therefore be considered. Indraratna et al. [35] studied two gradations of ballast material, and one of the results shows the influence of the typically low confining pressures seen in ballast on Poisson's ratio. This relationship, seen in Figure 10, indicates that Poisson's ratio decreases with increasing confining pressures. It was shown that the correlation is not linear, with changes in Poisson's ratio beyond 180 kPa (26.11 psi) being insignificant.

2.3.10 Settlement

Settlement of the track is largely the result of the vertical component of the plastic deformations that occur in the ballast as discussed in the previous section. To a lesser extent, long term settlement can also occur due to the subgrade. This discussion will focus on the contribution of settlement from the ballast, since ballast accounts for up to 76% of track settlement. [58]

A study involving one dimensional loading tests by Indraratna et al. [32] investigated the influence of differing load applications on the settlement of ballast. Specimens were either rapidly loaded or loaded in a stepped fashion. The maximum load in each case was the same, and it was held for 24 hours. The study found that the rapidly loaded specimens had larger initial settlements but small amounts of creep, while the specimens under stepped loading had much larger creep deformations. Regardless of loading conditions, the total, long term settlements of the specimen were nearly the same. This behavior can be seen in Figure 12. This series of tests also investigated the influence of water on settlement by flooding specimens with water when under load. This resulted in settlements of an additional 40% when compared to the dry specimens under the same load.

Settlement behavior of ballast under cyclic loading differs from that under static
loading. Each cycle of loading induces some amount of plastic strain and elastic strain. Several studies have shown that plastic strains accumulate at a diminishing rate with each cycle of loading, while elastic strains remain approximately constant with the number of cycles. The reduction in incremental plastic strains with increasing cycles is sometimes referred to as the work-hardening effect or the density increase effect. Eventually, permanent strains will be so small that the material behaves in a nearly perfectly elastic manner. One study found that this stabilization of permanent axial deformations occurs more quickly at induced deviator stresses below 90 kPa (13 psi) versus larger stresses up to 200 kPa (29 psi). Despite decreasing incremental plastic strains, railways can still experience large settlements since they can undergo millions of cycles between maintenance cycles. [32] [63] [64] [16]
2.3.11 Resilient Modulus

Cyclic loading is of particular interest when investigating railroad track. Each cycle of loading induces some amount of plastic and elastic strains on the ballast. Under constant cyclic stress conditions, the plastic strains increase at diminishing rates with each cycle. When plastic strains diminish such that incremental plastic strains are smaller than the elastic strains, the ballast essentially behaves elastically. Under these conditions, the ratio of the cyclic stress to the cyclic elastic strain is known as the resilient Youngs modulus, $E_r$, which is defined by:

$$E_r = \frac{\Delta \sigma_d}{\varepsilon_{1r}}$$

where $\Delta \sigma_d$ is the cyclic principle stress difference and $\varepsilon_{1r}$ is one half of the peak to peak elastic strain. [58]

The cyclic loading that the railway undergoes makes understanding resilient response of the ballast an important part of evaluating track performance. Various laboratory testing has been completed to understand various aspects of this. Several studies have shown that the duration of a load has little influence on the resilient behavior [25] [56]. Similarly, when the frequency of loading is within expected serviceable range, it has a very small impact on the resilient response [56]. Knutson and Thompson [43] investigated the resilient response of open-graded ballast aggregates in a laboratory setting. This study established that resilient modulus is not considerably affected by gradation, compactive effort, or stress history, provided that a stress resulting in a failure condition was not reached. The resilient modulus is, however, stress-dependent, showing a correlation to the stress level applied to the ballast.

A study by Indraratna et al. [34] investigated the effects of particle breakage on the resilient modulus. This study showed that particle breakage increases the resilient modulus regardless of the confining pressures applied. Figure 13 shows the
results from this study, which also confirms the stress-dependent nature of the resilient modulus, as described previously.

![Figure 13: Relationship between resilient modulus and effective confining pressure for different amounts of particle breakage [34]](image)

This result indicates that increased particle breakage decreases the void ratio and increases the interparticle contact area, which increases the resilient modulus. This study further provides a relationship between resilient modulus and bulk stress, or the sum of principle stresses, shown in Figure 14.

![Figure 14: Relationship between resilient modulus and bulk stress [34]](image)
2.4 Fouled Ballast

Finer materials introduced into the void spaces of ballast is known as fouling. The most widely accepted definition of fouling is any particle within the ballast that is smaller than 3/8” [58]. The largest source of fouling is ballast breakdown according to studies conducted by the Canadian Pacific Railroad [42]. This study was expanded at the University of Massachusetts to include many track conditions found in America [58]. Figure 15 shows the sources of fouling found for all site locations used in both studies. Fouling can have large influences over the behavior of ballast depending on the quantity and type of fouling. There are several methods by which fouling can be quantified, which are provided in the following section. Additionally, a discussion of geotechnical properties of ballast as they are affected by fouling is presented.

![Figure 15: Sources of Fouling [58]](image)

2.4.1 Quantification of Fouling

Ballast can be qualitatively classified as clean, partially fouled, or fully fouled, depending on the fouling content. An illustration of this can be seen in Figure 16. Often, a more quantitative method of describing fouling is desired for engineering purposes,
and several methods have been developed to do this. Selig and Waters (1994) proposed two commonly used indices, the fouling index \( F_I \) and the percentage fouling. The fouling index is based on gradation data, and given by

\[
F_I = P_4 + P_{200}
\]

where \( P_4 \) is percent passing the 4.75 mm (No. 4) sieve and \( P_{200} \) is the percent passing the 0.075 mm (No. 200) sieve. Related to \( F_I \) is the percentage fouling index, which is the ratio of the dry weight of material passing the 3/8” (9.5 mm) sieve to the dry weight of the total sample. These two indices have been shown to have a linear relationship [65] [38]. There are some limitations to these indices, as only a small number of fouling materials were used to create these empirical relations. Therefore, these indices cannot be reliably used when the specific gravity of the fouling material is considerably different from the ballast material.

![Figure 16: Schematic diagram of (a) clean, (b) partially fouled, and (c) totally fouled ballast [36]](image)

Another commonly used index is the percentage void contamination (PVC) proposed by Feldman and Nissen [19]. This index is the ratio of the total volume of re-compacted fouling material to the void volume between re-compacted ballast particles. This method more directly measures percentage of voids occupied by fouling, so it can be used for materials with different specific gravities. However, this index also has limitations as the fouling gradation is not accounted for and could therefore overestimate the fouling level. Additionally, the volumetric measurements can be
A newer measurement of fouling has been proposed by Indraratna et al. [38] to address some of the problems with the more commonly used indices described above. The relative ballast fouling ratio is defined by the equation

$$R_{b-f} = \frac{M_f \left( \frac{G_{sb}}{G_{sf}} \right)}{M_b}$$

where $M$ is the dry mass, $G_s$ is specific gravity, and the subscripts $f$ and $b$ designate fouling material and ballast, respectively. This ratio relates the solid volumes of the ballast and fouling material, and thus accounts for materials of different specific gravities better than other methods. It also more accurately assesses fouling with differing gradations. An additional advantage of this method is that it only requires three measurements: mass of the ballast, mass of the fouling material, and specific gravity of the fouling material.

An additional method of determining the extent of fouling has been developed specifically for clay fouling material. This method, called the void contamination index (VCI) is defined by

$$VCI = \frac{V_f}{V_{eb}} = \frac{1 + e_f}{e_b} \times \frac{G_{sb}}{G_{sf}} \times \frac{M_f}{M_b} \times 100\%$$

where $V_f$ is the actual volume of fouling material and $V_{eb}$ is the volume of ballast voids, $e_f$ is the void ratio of the fouling material, $e_b$ is the void ratio of the clean ballast, $G_{sb}$ is the specific gravity of the clean ballast, $G_{sf}$ is the specific gravity of the fouling material, $M_f$ is the mass of fouling material, and $M_b$ is the mass of clean ballast. This equation is intended to be a modification of the PVC which better accounts for variation in specific gravity between the ballast and fouling material. This method, however, does require more measurements than the relative ballast fouling ratio, which also accounts for specific gravity differences. [36]
2.4.2 Effects on Geotechnical Properties

Studies have shown that fouling has generally negative impacts on the geotechnical properties of ballast. These include loss of drainage, prevention of particles moving through the ballast, growth of vegetation, reduction in track resiliency, compromised ability to withstand vertical, lateral, and longitudinal forces, increased rate of ballast deterioration, and poor durability following maintenance [18] [58] [68].

One of the most significant issues with fouling is the decrease in permeability resulting in poor drainage of water away from the track. Under loading, this can result in significant increases in pore pressure, which can decrease the shear strength and stiffness of the ballast [1] [2] [30] [33] [64]. It has also been found that reduction in permeability is a major cause of track deterioration, resulting from issues such as hydraulic erosion, subgrade attrition, reduced stability, and ballast deterioration [37] [64].

Testing has been done to further investigate the effects of fouling on the shear strengths of ballast. Several studies have found that the addition of coarse-grained fouling can increase the strength of ballast when in the dry condition. This happens because the soil matrix is more broadly graded, but is only valid when the main structure is still formed by the ballast [58] [40]. Somewhat in contrast to this, Duong et al. [16] observed that an addition of 10% fines did not significantly increase the shear strength. However, this study did find that increased fines in an unsaturated state does increase strength due to increased suction generated by the fines. As the fouled ballast approached the saturated state, added fines decreased shear strengths significantly. Similar findings are reported by Ebrahimi [17], Kim et al. [41] and Huang et al. [30].

The above studies would seem to suggest that fouling in ballast, provided it is not saturated, may be beneficial. However, despite the apparent improvements in strength, the negative impacts of reduced track resiliency, increased ballast deforma-
tions, and compromised ability to withstand loads outweigh this. Increased fouling also leads to increased maintenance, which often leaves the ballast in a looser state [35].

It is argued that there is a threshold of fouling content at which the behavior is completely changed. There is no single ratio that defines this threshold, suggesting that the nature of the soil plays a role in the behavioral changes of ballast [16]. This brings up the importance of the characterization of the fouling by grain size and resulting behaviors. Gravel and sand-sized fouling will effect the ballast behavior differently than clay and silt-sized fouling particles.

Limited studies have been done specifically with clay fouling. While coarse grained fouling can increase strength, clay fouling typically reduces shear strength because the addition of clay reduces the internal friction angle of the ballast. In one study [36], it was found that beyond a Void Contamination Index of 25%, peak deviator stresses decreased significantly with increase fouling. Below 25% VCI, there is a distinct drop in the post peak strength of the material. The volumetric strain behavior of ballast is affected by fine-grained fouling as well. Testing showed that small amounts of clay fouling can reduce the rate of dilation of clean ballast under load, but the complimentary significant losses in shear strengths negates any benefit that might be seen from reducing dilation. Increasing fouling content with clay also reduced compression, likely because the void spaces in the ballast are filled with clay. However, some smaller amounts of clay fouling can act as a lubricant between ballast particles, thereby increasing compression.

Generally, greater performance issues are encountered when fouling contains fine grained particles like silt and clay. This type of fouling will decrease drainage, causing the ballast to hold water, a critical contributor to maintenance issues. The presence of water increases the likelihood of ballast deterioration through hydraulic erosion, subgrade attrition, and decreased stability through particle lubrication. This lubri-
cation can also cause problems with maintaining track geometry since it weakens the structure of the ballast. Additionally, when silts are present, or clay is combined with coarse grained fouling, an abrasive slurry can form, which hastens the deterioration of the ballast. Often, full replacement of the ballast is necessary when fine fouling is present because it coats the ballast particles making it difficult to separate. [58]

2.5 Modeling

The understanding of ballast behavior as well as the evaluation of track performance can be improved with models. Such improvements can help reduce maintenance costs and improve operation and safety. Before models were used, stresses and displacements in ballast were determined using empirical methods. However, since the 1980s, increased train speeds and freight loads prompted more precise methods, leading to the increased use of computer driven models. Generally, there are two broad categories of numerical models. One type incorporates exact solutions for a multilayer elastic system and the other type uses finite element analysis. [62]

Early models include MULTA, PSA, and ILLITRACK developed by Selig et al. in 1979 [60]. ILLITRACK is a finite element model which is composed of two two-dimensional models, one for the transverse direction and one for the longitudinal direction. The following year, Chang et al. [9] provided improvements to the MULTA system with the GEOTRACK model. GEOTRACK is a fully 3D and multilayered model. It considers axle loads, rail and properties, ballast and underlying layer properties, and track geometry. The model uses these inputs to determine track deflections and modulus, as well as provide estimates of stresses and deformations of the ballast, subballast, and subgrade layers. Models such as these are useful for understanding the overall track performance. [62] [58]

An increased understanding of behavior specific to ballast can be better understood through the modeling of laboratory tests. Box tests are perhaps the most
extensively modeled ballast laboratory test. Lu and McDowell [46], for instance, used a discrete element model. Later, Bennett et al. [7] developed a finite element model using PLAXIS software to model box tests, which is of particular interest because of the materials modeled. Laboratory tests were performed on fairly rounded quartzite ballast with clay fouling, which was selected because the greatest behavioral changes were expected from wet clay fouling. This modeling showed that plastic settlements reached a minimum and became constant after 1000 cycles. By calibrating the model, a plot of elastic modulus as percentage of fouling material, as seen in Figure 17, was developed. This model also showed a trend of increased elastic deformation with increased fouling. This trend had one exception in the 40% fouling case, which had a high degree of plastic deformation. Models such as this provide a much better understanding of ballast mechanics.

![Figure 17: Elastic modulus as a function of percentage fouling [7]](image)

Discrete element models developed by Lim and McDowell [45] investigated individual grain crushing strength tests, oedometer tests, and box tests. This model used agglomerates of balls to model the ballast particles. The oedometer simulation showed higher compressibility than the laboratory tests after yielding, but lower compressibility when higher stress levels were reached. The higher compressibility was attributed to the fact that the internal voids of the agglomerates, when fractured, added to the void ratio of the bulk material. The model also delivered yield stresses lower than that of real ballast. This discrepancy was attributed to the agglomer-
ates being spherical in nature. While this model provided a fairly good simulation, improvements could be made.

Another type of model, which investigates the density packing of binary granular mixtures, is not specific to ballast, but is of particular interest to this research. It was observed by Selig and Ladd [61] that the relative density and the range of possible void ratios are essential components in estimating the behavior of granular materials. These properties are influenced by the particle packing and grain size distribution of soils. Empirical methods were developed for estimating maximum packing density (minimum void ratio) such as the AASHTO “correction factor” method which is applicable to soils with 70% or less gravel-size particles [4] and the Humphres method for solids with different particle sizes [31]. Analytical methods were initially developed by de Larrard [14], and later Chang and Meidani [10] developed improved models for predicting stress-strain modeling of sand-silt mixtures. [11]

The original model by Chang and Meidani [10] uses a dominant network concept for binary mixture packing. It was later expanded by Chang et al. [11] [12] to estimate maximum and minimum void ratios of sand-silt mixtures with different particle sizes. These models showed that an increase in fines increases the packing density until the voids of the coarser grained particles of filled. Beyond this point, the packing density decreases as the larger particles are pushed apart until they become isolated inclusions in a fine-grain dominant matrix. Figure 18 shows this relationship for steel shots. [11] [12]

These models connected void ratio to packing structure. It was found that more angular particles have generally higher minimum void ratios. The packing efficiency increases as the size ratio of coarse to fine grains approaches seven, and then decreases rapidly. The range of possible minimum void ratios of binary mixtures as related to fines content was determined, as shown in Figure 19. The variation in void ratio shows that with respect to fines, the curves are V-shaped and that the packing density
Figure 18: Characteristics of Packing Density due to fines content for steel shots [11] is largely dependent on the size ratio of coarse to fine particles. And while the results shown are from steel shots, this behavior was the same for sand-silt mixtures, despite having different shape characteristics. The relationship between maximum and minimum void ratios was determined through derivations, and it was shown to be linear. These models developed were used to replicate 120 samples of varying fines content with a difference of only 2% between predicted and measured void ratios, providing much insight into the packing of binary mixtures. [11] [12]

2.6 Previous Work

The research outlined in the following sections largely builds upon the previous work done by Hamed Kashani for his Ph. D. dissertation research at the University of Massachusetts, Amherst. This research encompassed several studies, two of which will be discussed within this section to help provide justification for the research herein.

Ballast box tests were performed to investigate heavy axle loading conditions on ballast and the resulting elastic and plastic settlement behavior and ballast degradation. The box used new dimensions, with an increase in size employed to reduce
boundary condition effects. A highly angular Connecticut granite ballast was used in clean, moderately fouled (15%), and highly fouled (30%) conditions. Five different water content conditions were tested; dry, field capacity, two points between them, and saturated. The ballast was compacted in lifts, ensuring that each test had equivalent amounts of ballast. The fouling was added to each lift to ensure even distribution throughout the sample. A section of tie was embedded 4 inches (10.16 cm) into the top layer with a LVDT used to record differential settlements between the box and tie. Heavy axle loading of 60 kips (270 kN) was determined by calculating the Equivalent Wheel Load using a GEOTRACK model. This load was applied for up to 2.5 million cycles, which included a 25%, 50%, and 75% increase in loading during the saturated phase of the test. [40]

Results from the box tests were verified with field measurements, confirming the validity of the new box dimensions. The tests results indicated that when fouling and water content are high under heavy traffic conditions, there is a great risk for high

Figure 19: Characteristics of void ratio versus fines content for sand-silt mixtures [11]
levels of settlement. Under saturated conditions, the settlement can increase by 50 times over the field capacity conditions. It was shown that for clean ballast, settlement changes were not significant across water contents from dry to field capacity. Increasing moisture led to higher settlement rates in moderately fouled ballast than heavily fouled ballast. The settlement is mostly affected by changes in fouling content at field capacity and saturated conditions. The study further showed that an increase in load up to 75% did not greatly affect the settlement. In saturated conditions, the elastic response was greater for fouled ballast with an increase of accumulated loading cycles. The ballast strength of moderately and heavily fouled ballast is affected more by compaction from traffic than by degradation. Additionally, for these levels of fouling, a 3% increase in water content can triple the elastic settlements observed. Finally, this testing determined a linear relationship between elasticity and traffic load. [40]

An additional study by Kashani et al. [39] further investigated the same ballast and fouling material using isotropically consolidated drained (CIDC) triaxial testing. The fouling contents and water contents tested were the same, though no saturated tests were performed. The prepared triaxial specimens had a 10-inch (25.4-cm) diameter, with ballast and fouling compacted in lifts, ensuring uniform density, similar to the preparation of the box tests. Tests were run using confining pressures of 5 psi (34.5 kPa), 10 psi (68.9 kPa) and 15 psi (103.4 kPa). Static loading was applied using a strain control rate of 0.04 in/min (0.102 cm/min) which is slow enough to prevent increased pore pressures for the wetted tests. Since the tests were not saturated, volumetric behavior was estimated by using circumferential string potentiometers placed at the midsection and quarter sections of the specimen.

The results of the triaxial testing provided stress-strain behavior, Mohr-Coulomb strength properties, elastic modulus, volumetric strain behavior, and degree of particle breakage. The study concluded that increasing water content linearly decreased the
maximum shear strength of the ballast. Strength loss was not observed with increased fouling at the same water content. Increased fouling resulted in increased elastic modulus, while increased water linearly decreased the elastic modulus. The friction angle also increased with increased fouling, with little noticeable effect from water content. However, increased water content decreased the friction angle at a rate which decreased with increased fouling. Increased confining pressures resulted in increased particle crushing, but this effect was reduced with increasing water content. Finally, it was found that overall, fouling had a larger influence than water content in influencing ballast strength and volumetric strain rate during dilation. [39]

These studies provide a better understanding of ballast behavior under laboratory testing conditions. Correlations of the box test to field measurements also validate their effectiveness in simulating the real-world conditions. However, this testing is limited to one type of ballast and fouling. Further investigation using the same testing methods but different materials could provide a broader understanding of fouled ballast behavior. Specifically, ballast of different levels of roundness or abrasion could be investigated. Fouling from sources other than ballast breakdown could also be investigated.
CHAPTER 3
TESTED MATERIALS AND INDEX PROPERTIES

As previously outlined, one of the objectives of this dissertation is to provide a better understanding of ballast behavior based on different particle angularities and fouling materials. Therefore, the selection of appropriate ballast and fouling is critical for the success of the research. This work builds on a previous study which used highly angular ballast and breakdown fouling. To compare to these materials, this study uses abraded ballast and different fouling mixtures. Further details on these materials are provided in the following sections.

3.1 Description of Ballast and Fouling Materials

Previous work conducted by Kashani [40] was performed on an angular granite ballast, freshly quarried in Connecticut. The fouling material used was granitic stone dust taken from the same quarry. This would be representative of breakdown fouling, or fouling generated from ballast abrasion. Therefore, the ballast and fouling come from the same parent rock and are composed of the same material at different grain sizes.

An abraded ballast of basaltic origin is used for this study so that particle angularities can be compared. The abraded ballast is sourced from a track in Shelburne Falls, Massachusetts where the owner of a line performed a full replacement of ballast. An image of the site, in Figure 20, shows the site that the ballast is sourced from. The darker spent ballast can be seen adjacent to the newly placed ballast below the track. Figure 21 shows a close-up image of some particles taken from the ballast sample. A variety of particle sizes are shown. It can be seen that the larger particles have some corners which have been rounded off. The smaller pieces shown tend to be more angular as they are likely the broken-off fragments of the larger particles. Using this material coming directly from in-use track provides several advantages for this
study. First, the ballast has been naturally abraded from real traffic loading. Second, the fouling consists of both breakdown fouling and fouling which has penetrated the ballast from the subgrade or from the top by water or windblown forces. Both of these factors help to provide a testing material which should provide very realistic results. For the purposes of testing, specific quantities of fouling are required for each test. Therefore, all material collected from the site was processed by mechanical sieve shaker and all particles passing the 3/8” (9.5 mm) sieve were removed as fouling from the ballast. When building specimens for testing, the fouling is reintroduced in the desired amounts.

The first set of laboratory tests use the abraded ballast with the natural fouling material alone. The second set of tests is performed using the same ballast as the first set, but the natural fouling material is mixed with Prestige clay, which is an industrial grade kaolinite clay. A mixture of 45% Prestige to 55% natural fouling by mass was selected based on Atterberg Limit testing, which is discussed in detail in a later section. By mixing with the natural fouling, this material provides a plastic fouling which could reasonably exist in the track.
The two ballasts and three fouling materials used in this study and the previous work allow for two important comparisons to be made. The first is the effect of ballast particle shape or abrasion level on ballast performance. The second is the influence of different fouling materials on ballast performance. The fouling materials range from predominantly sand sized grains to predominantly clay sized grains. Comparison of tests in similar fouling conditions but with differing fouling material will provide insight to the effects of not just the amount of fouling, but the makeup of it as well.

3.2 Grain Size Distribution

Railroad ballast is typically selected to be of relatively uniform grain size, which provides large voids and facilitates drainage away from the track. Several different standard grain size distributions are given by the American Railway Engineering and Maintenance-of-Way Association [3] which are dependent on the use case of the track. Two samples of ballast and fouling were taken from the track in Shelburne Falls, MA for testing. The sieve analyses were performed in general accordance with ASTM
In ballast, any material passing the 3/8” (9.5 mm) sieve is considered to be fouling. Using the percentage fouling measure, it was found that the ballast was 23% fouled in its natural state. The grain size distribution for each of the two ballast samples, not considering the fouling, is given in Figure 22. The ballast most closely meets AREMA #4 gradation, the limits of which are also presented in the figure. The grain size distribution curves indicate that the ballast experienced breakdown under loading, causing the curves to fall outside the recommended gradation. Observationally, there are many small particles which are large enough to be classified as ballast, but are likely the broken fragments of other larger particles. Generally, the larger ballast pieces are more rounded, while the smaller ones are more angular. Figure 23 provides the sieve analysis for the fouling material alone. Using these data and the Unified Soil Classification System (USCS), it can be seen that the majority of the fouling is
composed of sand size particles. Just 9% of the fouling is fine gravel retained on the #4 sieve, and 18% is silt and clay size particles passing the #200 sieve.

Figure 23: Sieve analysis results for natural fouling material.

3.3 Mill Abrasion

The parent rock of the aggregates used in railroad ballast need to have high strength and durability. An important basis for evaluating these properties are mechanical abrasion tests. One such test is the mill abrasion test, originally developed by Selig and Boucher [59]. Mill abrasion tests were performed for both the angular granite and abraded basalt ballasts. The ballast is first thoroughly washed and dried before being placed into a mill abrasion drum with distilled water. The drum is sealed and placed on rollers which turn at 34 rpm for 10,000 revolutions. The material is then wet sieved through the 3/8” (9.5 mm) and #200 sieves and the retained material is dried and massed. From these data, several parameters can be calculated; Mill Abrasion Value (MA), Broken Material Generated (B) and Proportion of Broken Material finer...
than # 200 (P), which are given as follows:

\[
MA = \left(\frac{W_t - W_1 - W_2}{W_t}\right) \times 100\% 
\]

\[
B = \left(\frac{W_t - W_1}{W_t}\right) \times 100\% 
\]

\[
P = \left(\frac{W_t - W_1 - W_2}{W_t - W_1}\right) \times 100\% = \frac{MA/B}{100}\% 
\]

where \(W_t\) is the total initial mass, \(W_1\) is the mass of material retained on the 3/8” (9.5 mm) sieve, and \(W_2\) is the mass of material passing the 3/8” (9.5 mm) sieve. Because the ballast from Shelburne Falls is already abraded, it is expected to have a lower Mill Abrasion Value than the granite. Two trials were performed on the abraded ballast, and three were performed on the granite. The results in Table 3 are the averaged parameter values from the individual trials.

Table 3: Mill Abrasion test results for granite and abraded ballasts.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Abraded</th>
<th>Granite</th>
</tr>
</thead>
<tbody>
<tr>
<td>MA (%)</td>
<td>3.4</td>
<td>4.4</td>
</tr>
<tr>
<td>B (%)</td>
<td>3.5</td>
<td>4.8</td>
</tr>
<tr>
<td>P (%)</td>
<td>96.7</td>
<td>91.2</td>
</tr>
</tbody>
</table>

As might be expected, the abraded basalt has a lower Mill Abrasion Value and less broken particles generated than the angular granite. This is largely due to the fact that the ballast is already abraded from loading in the track, but basalt is also a harder rock than granite, which may further contribute to the smaller amounts of material. The abraded basalt ballast also has a larger proportion of broken material passing the # 200 sieve. Again, because of the previous abrasion, it is less likely that larger pieces will break off of the abraded basalt. The angular granite, on the other hand, has corners that are more likely to break off, creating larger particles. It is important to keep this difference between the ballasts in mind when evaluating later laboratory test results.
3.4 Atterberg Limits of Fouling

To compare the effects of different fouling types on the ballast behavior, it is important that each has different properties. The fouling from previous testing was a granitic stone dust, representative of breakdown fouling. The first fouling material for this study is the natural fouling taken from the track, which is a mixture of both breakdown fouling and other sources. It was desired that the final fouling material have a component that behaves plastically. There have been some studies in the past that have achieved this by testing ballast with pure clay fouling [36], but a case such as this is unlikely to be encountered in the field. Fouling with high clay contents are very site specific, but worth exploring. It was decided that the final fouling material would be a mixture of the natural fouling taken from the track and Prestige clay. Using a mixture takes advantage of providing more realistic results driven by the natural material, while also providing plastic behavior from the clay.

In order to determine an optimal mixture of natural fouling to clay, Atterberg Limits tests were performed on several different mixes in general accordance with ASTM D4318-10 Liquid Limit, Plastic Limit, and Plasticity Index of Soils [5]. Plastic limit (PL) and liquid limit (LL) tests were performed on both the pure natural fouling and pure Prestige clay to determine a baseline. Then, mixtures of 25%, 35%, 50%, 60%, and 70% Prestige clay by mass were prepared and tested. It should be noted that Atterberg limits can only be performed on material passing the #40 sieve, so larger particles had to be removed from the natural fouling. Additionally, on the pure natural fouling tests, only material passing the #200 sieve is used, as it was found that the tests could not be completed using all material passing the #40. This is because not a large enough portion of the total material sample passed the #200, so there is not enough plasticity to perform the tests. The results obtained from the tests are plotted on the Casagrande plasticity chart given in Figure 24.

It can be seen from these results that the natural fouling material passing the
Figure 24: Atterberg limits results for several natural fouling and Prestige clay mixtures.

#200 sieve is classified as a silt and the Prestige Clay is a lean clay of high plasticity. The first fouling mixture of 25% Prestige plots on the line between silty clay and clay. The other data points show that as the clay content increases, the liquid limit and plastic limit steadily increase. Based on these results, a mixture of 45% Prestige was ultimately selected. This mixture was not tested for Atterberg limits, but based on the consistency of the results from these tests, it can be determined that the mixture has properties $LL = 31$ and $PI = 12$. This selection was made for several reasons. First, with a $PL = 19$ this mixture will have a predominantly plastic response during testing. Second, the mixture is still predominantly natural material by mass, making this a reasonably realistic mixture. Third, based on mixing models by Chang et al. [12] this mixture should provide the best packing of the two materials. This mixture is henceforth referred to as Prestige mix fouling in this dissertation and is used in all pertinent tests.
3.5 Petrology

The ballast collected from Shelburne Falls, MA was identified as a basalt traprock in the field. However, it is difficult to identify what the fouling material consists of. An X-ray Floressence (XRF) Spectrometer can be used to identify the minerals contained within the material. Three samples were sent for testing: ballast, “coarse” fouling, and “fine” fouling. The “coarse” fouling is that retained by the #10 sieve, and the “fine” fouling is all the material passing. The XRF results show that the mineral contents are extremely similar for the ballast and the “coarse” fouling, indicating that this portion of the fouling material is primarily broken fragments of the ballast particles. The “fine” fouling has a much higher silica content, indicating that other sources are contributing to the fouling. This supports the claim that the natural fouling material is comprised of both breakdown fouling and fouling that has infiltrated from either the subgrade or by transport by wind or water. The XRF testing also included loss on ignition (LOI) tests, measuring the presence of organics. The different materials had LOI of 2.9-7.4%, which is likely comes from the wooden fragments in the material. When the ballast was removed from the track, many of the wooden ties were also replaced. During this process, many of the old ties were crushed and broken by heavy equipment, leaving behind small wood fragments found throughout the material. The XRF results which breakdown the mineral components are given in Table 4.

Table 4: Percentage of minerals determined by XRF tests on ballast and fouling samples.

<table>
<thead>
<tr>
<th>Sample</th>
<th>SiO₂</th>
<th>TiO₂</th>
<th>Al₂O₃</th>
<th>Fe₂O₃*</th>
<th>MnO</th>
<th>MgO</th>
<th>CaO</th>
<th>Na₂O</th>
<th>K₂O</th>
<th>P₂O₅</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample 1 Ballast</td>
<td>52.58</td>
<td>1.03</td>
<td>14.11</td>
<td>13.59</td>
<td>0.23</td>
<td>5.63</td>
<td>7.52</td>
<td>4.41</td>
<td>0.64</td>
<td>0.13</td>
<td>99.87</td>
</tr>
<tr>
<td>Sample 2 Coarse</td>
<td>54.12</td>
<td>1.02</td>
<td>14.08</td>
<td>12.13</td>
<td>0.21</td>
<td>5.29</td>
<td>6.83</td>
<td>4.51</td>
<td>0.33</td>
<td>0.13</td>
<td>100.25</td>
</tr>
<tr>
<td>Sample 3 Fine</td>
<td>70.14</td>
<td>0.42</td>
<td>9.33</td>
<td>0.86</td>
<td>0.14</td>
<td>1.18</td>
<td>1.05</td>
<td>2.03</td>
<td>1.83</td>
<td>0.12</td>
<td>99.85</td>
</tr>
</tbody>
</table>
3.6 Quantification of Fouling

In the United States, fouling is most commonly measured by the percentage fouling index, which is given by the mass of fouling over the total mass of the sample. The previous work by Kashani [40] used this measure to determine fouling conditions. The laboratory box tests and triaxial tests were performed at 0%, 15%, and 30% fouling. This study, building on the previous work, used the same method of measurement and prepared the same fouling conditions when testing the abraded ballast and natural fouling material. However, after beginning to work with the Prestige mix fouling, it became apparent that there are some issues with the percentage fouling methodology.

Due to the very low unit weight of the clay, the Prestige mix fouling has twice the volume of the natural fouling alone. When building 30% fouled triaxial specimens, the volume of the fouling exceeded the volume of the voids in the ballast, so the samples physically could not be built to the required specifications for the test. This resulted in a reexamination of the percentage fouling methodology of quantifying the fouling condition of ballast.

The percentage fouling measure was developed by empirical methods, and was limited to a small number of fouling materials, which were primarily representative of breakdown fouling. This method only considers the mass of the materials, so its effectiveness diminishes when the fouling has a significantly different specific gravity from the ballast. Because this is the case with the abraded ballast and Prestige fouling mix, the percentage fouling method is not well-suited for quantifying fouling in these tests.

Feldman and Nissen [19] have previously noted this issue with the percentage fouling method. They proposed a different measure called percentage void contamination, given by
\[ PVC = \frac{V_2}{V_1} \]  

where \( V_1 \) is the void volume of compacted ballast and \( V_2 \) is the volume of compacted fouling material. After some consideration, it was decided that PVC would be an appropriate measure for this situation. This methodology would allow specimens to be prepared to have the same volume of fouling rather than the same mass. This would allow for specimens to be reconstituted in such a way that the results can still be comparable to previous tests.

Using this measure, it was found that the ballast with 15% and 30% natural fouling had equivalent PVC values of 35.6% and 86.3% respectively. Therefore, the Prestige mix tests were prepared to match this. Conveniently, the volume of the Prestige mix is twice that of the natural fouling alone, so the correct PVC can be achieved by using half the mass of fouling. The percent fouling and PVC values for each type of fouling at the three different fouling contents is given in Table 5. While the PVC tests are not truly 15% and 30% fouled, they are directly compared to the natural fouling tests which are. Therefore, for the purpose of clarity, these terms will be used to describe the tests in this dissertation.

Table 5: Values of % fouling and corresponding PVC values for each fouling material.

<table>
<thead>
<tr>
<th>Fouling Type</th>
<th>0%</th>
<th>15%</th>
<th>30%</th>
</tr>
</thead>
<tbody>
<tr>
<td>% Fouling</td>
<td>PVC</td>
<td>% Fouling</td>
<td>PVC</td>
</tr>
<tr>
<td>Granitic Stone Dust</td>
<td>0</td>
<td>0</td>
<td>15</td>
</tr>
<tr>
<td>Natural Fouling</td>
<td>0</td>
<td>0</td>
<td>15</td>
</tr>
<tr>
<td>Prestige &amp; Natural Fouling Mixture</td>
<td>-</td>
<td>-</td>
<td>8.1</td>
</tr>
</tbody>
</table>
Consolidated drained triaxial tests were conducted on reconstituted ballast samples to determine strength properties and deformation behavior of ballast under different fouling and moisture conditions. The following sections detail the specimen preparation, including determination of fouling and water content, and the testing procedures.

4.1 Specimen Preparation

Consolidated Isotropic Drained Compression Triaxial Tests (CIDC) were performed in general accordance with ASTM D7181-11 [5]. 10-inch (25.4-cm) diameter specimens were prepared in eight lifts, each with a thickness of 2.5 inches (6.35 cm). In each lift, the ballast was placed first, followed by the appropriate amount of fouling. The material is then tamped to promote mixing and to achieve the proper density. It is important to note that the quantity of ballast is the same in every test, with only the amount of fouling changing. As a result, the ballast density remains the same while the total sample density increases with increasing fouling.

Once the specimen is prepared and the testing cell is fully assembled, the moisture content can be considered. To simulate field conditions, the water contents are based on field capacity, which is the maximum amount of water that is held by the material in a free draining condition. The field capacity is determined by preparing a sample of ballast and fouling in a bucket with drain holes. The specimen is covered in water, allowed to drain, and the water content is taken. Based on these results, triaxial specimens are tested in the dry condition, at half of field capacity, and at field capacity. Water is added to the top of the sample and a minimum of 16 hours is allowed to pass so that the water has sufficient time to fully penetrate into the sample.
A low cell pressure of 2-3 psi (13.8-20.7 kPa) is used to maintain the specimen shape until the time of testing.

Since the tests are performed in undersaturated conditions, the volume change of specimens cannot be directly measured by the flow of water in and out of the sample. Instead, three string potentiometers are placed circumferentially on the mid-point and quarter-points of the specimen. The measurements taken are used to calculate radial strain, which in turn are used to estimate volume change, which is discussed further in the next section. An example of the triaxial test set up is shown in Figure 25.

![Triaxial test setup in load frame. Instron control panel is shown to the left; the cell pressure control panel is shown to the right.](image)

Figure 25: Triaxial test setup in load frame. Instron control panel is shown to the left; the cell pressure control panel is shown to the right.

### 4.2 Testing Procedure

The triaxial test specimens are loaded using a 100-kN (22.5-kip) static capacity actuator operated by an Instron 8501 controller. The test parameters used are taken
from a study by Aursudkij et al. [6] which found that specimens loaded past 12% strain had reached and sustained peak strength. This study used the same strain rate of 0.04 inches/min (1 mm/min). Tests were run up to 12% strain or were stopped when the bulging specimen risked touching the sides of and damaging the testing cell. Confining pressures of 5, 10, and 15 psi (34.5, 69.0, and 103.4 kPa) were applied to replicate the characteristically low confining pressures of railroad track. All data were collected using a DATAQ Instruments DI-710 data acquisition box and WinDAQ software. Data was recorded at a rate of four readings per second per channel.

4.3 Triaxial Test Results

The drained triaxial tests were used to determine the strength properties, stress-strain behavior, and volumetric strain behavior of ballast under different fouling and moisture conditions. The following sections provide details which illustrate that the abraded ballast achieved similar strengths to the angular granite ballast, but had increased susceptibility to deformations. The results also show that the introduction of plastic fines into the ballast reduce strength and further increase susceptibility to deformations, making it particularly problematic for ballast performance.

4.3.1 Stress-Strain and Volumetric Strain Behavior

The ballast stress-strain curves indicate two distinct behaviors, which are controlled by both fouling content and moisture content. The majority of tests have behavior typical of loose sand, where strength increases at a diminishing rate throughout the test. The example curves from the clean, dry tests given in Figure 26a are representative of this behavior. It can be seen that the granite curves typically reach a maximum strength before reaching 5% axial strain, and that strength is maintained. The abraded ballast curves show a more gradual gain in strength, usually not leveling off until after 8% axial strain is reached. However, by the end of the tests, both
(a) Typical stress-strain behavior for clean and moderately fouled ballast. Example curves taken from clean tests in the dry condition.

(b) Typical stress-strain behavior for heavily fouled ballast. Example curves taken from 30% fouled tests in the dry condition.

Figure 26: Typical stress-strain curves.

ballast types achieve similar strengths.

Some of the tests in the heavily fouled condition have a stress-strain behavior that is more similar to dense sand, where a peak strength is reached, followed by strain softening. Figure 26b shows the 30% fouled tests in the dry condition which
display this behavior. Again, the angular granite ballast gains strength more quickly than the abraded ballast, reaching the peak strength at a lower axial strain. The peak strengths are similar for both ballasts, as are the difference between peak and residual strength. It should be noted that the 30% fouled granite was able to achieve peak strengths even with the addition of water, but the abraded ballast with natural fouling.
reverted back to the behavior of loose sand when wet. This loss of peak strength is shown in the two plots in Figure 27, which show 30% fouled tests in the dry and 50% field capacity moisture conditions for abraded ballast with natural fouling. This figure also shows results from the abraded ballast with Prestige mix fouling. It can be seen that despite having the same volume of fouling as the other tests, no peak strength is achieved, even in the dry condition.

The stress-strain and volumetric strain of each ballast and fouling combination can be compared to draw further comparisons between their behaviors. Figure 28 shows the stress-strain curves and the volumetric strain curves for all triaxial tests performed at 30% fouling at half of field capacity. These tests were chosen as an example for discussion because they provide a good representation of behavior across most of the tests. For further information, Appendix A contains the stress-strain and volumetric strain curves for all tests performed.

From the example curves, it can first be observed that the granite ballast achieves a peak strength, followed by strain softening. The abraded ballast with natural fouling does not have a strong peak with the addition of water, but the final strengths reached are similar to that of the granite. The addition of Prestige to the fouling with the abraded ballast leads to a drop in strength across all tests. Additionally, observation of the stress-strain curves show that the modulus of the abraded ballast is lower than that of the granite. The modulus drops further when the abraded ballast is fouled with the Prestige mix fouling. Quantification of the modulus and methods of its measurement for each test is provided in the following section.

The volumetric strain curves use geotechnical conventions, meaning positive strains indicate compression while negative strains indicate dilation. The plots show behavior typical of ballast, with each test undergoing an initial contraction, which is typically followed by a much larger dilation. The abraded ballast undergoes higher levels of contraction than the granite ballast. In general, the addition of Prestige to the foul-
Figure 28: 30% fouled triaxial tests at half of field capacity. (Larger versions of these plots are available in Appendix A)

...ing causes the abraded ballast to contract even more. It can be observed that with increasing confining pressure, the amount of contraction generally increases for each...
ballast and fouling combination, as does the point at which contraction ends and dilation begins. In some rare cases, such as the 5-psi (34.5-kPa) test with Prestige mix fouling, the specimen never reaches dilation.

![Graphs showing maximum contraction for triaxial tests performed on granite ballast with breakdown fouling and abraded ballast with natural and Prestige mix fouling.](image)

Figure 29: Maximum contraction for triaxial tests performed on granite ballast with breakdown fouling and abraded ballast with natural and Prestige mix fouling.

When considering the volumetric behavior of the ballast in different fouling conditions, it can also be useful to look at the maximum contraction of the specimen and
Figure 30: Poisson’s Ratio vs axial strain at which maximum contraction occurs.

at which axial strain that point is reached. Figure 29 presents these data points for all of the triaxial tests performed in this study, as well as the granite tests performed as part of the previous work. It can be seen that the granite ballast has very consistent volumetric behavior, with the maximum contractions all occurring before 2%
axial strain is reached. Additionally, all the maximum contractions fall within about 0.5% and 1.5% volumetric strain. The abraded ballast with the natural fouling has a bit more variability, but is still has fairly consistent maximum contractions between 1% and 2% volumetric strain, all occurring within the first 5% axial strain. In contrast, the abraded ballast with Prestige mix fouling tests have maximum contractions ranging from 1% to 5% volumetric strain occurring at axial strains from 1% to 12%. This show a wide amount of variation in the behavior of this material, making it potentially difficult to assess in the field.

The Poisson’s ratio was also calculated for each test at the point of maximum contraction. It can be seen in Figure 30 that the two ballasts have similar Poisson’s ratios in the clean condition. These values are all around 0.2 which is a typical value for a loose sand or silt [26]. With the addition of fouling, the Poisson’s ratio of the granite drops slightly, while it increases slightly for the abraded ballast. It can also be seen that the Prestige mixture fouling further increases the Poisson’s ratio of the ballast and fouling matrix. Saturated clays typically have Poisson’s ratios of 0.4 to 0.5 [26], so the influence of the clay is apparent with these rising values. These plots also further demonstrate how the addition of Prestige to the fouling material contributes to a prolonged period of initial contraction in the tests.

4.3.2 Strength Properties

The strength properties of the ballasts can be obtained from the data collected during the triaxial tests performed. Ultimate strengths are determined by the maximum deviator stress measured during each test. Mohr-Coulomb failure envelopes can be developed for each combination of fouling and moisture content since each was tested at three different confining pressures. Since ballast is granular, it was assumed that there is no cohesion in these tests, so each failure envelope passes through the origin. The friction angles and ultimate strengths are presented in Table 8 which can be
The friction angle results are also shown graphically in Figure 31. Both the Connecticut granite and abraded ballast with natural fouling see increased friction angles with increasing fouling content, regardless of water condition. The abraded
ballast with Prestige mix fouling sees this same trend in the dry state, but increased fouling contents do not increase the friction angle when water is added. It can be seen that for any ballast and fouling material, at any given fouling content, the addition of water generally reduces the friction angle. In the case of the abraded ballast, however, the specimens show an increase in friction angle from dry to 50% field capacity, followed by a decrease from 50% field capacity to field capacity.

Typically, the friction angle of the granite and the abraded ballast are comparable, but the abraded ballast most often shows slightly higher values. However, the addition of Prestige clay causes a drop in the friction angle. At 15% fouling, Prestige mix fouling results in friction angles that are $1.8^\circ$–$3.5^\circ$ lower than their counterparts with natural fouling alone, and the 30% fouled tests have friction angles that are $2.5^\circ$–$6.8^\circ$ lower.

The ultimate strengths typically follow the same trends as the friction angles described above, since the friction angle is partially determined based on the ultimate strengths of the corresponding tests. It is useful to also look at the strengths in terms of water content. Figure 32 shows the ultimate strengths for all tests performed on the abraded ballast at 30% fouling, including both natural fouling and Prestige mix fouling. This is representative of most tests. It can be seen here that the ultimate strengths at a given confining pressure linearly decrease with increasing water content. The rate of strength decrease is usually larger for the Prestige mix fouling as compared to the natural fouling alone. There are no apparent trends of the rate of strength loss to either fouling content or confining pressure. It should be noted that there are some cases not shown that have a slight linear increase in strength, such as clean tests with 15 psi (103.4 kPa) confining pressure.
4.3.3 Modulus

The elastic modulus of the ballast can also be determined with the triaxial test data. Typically, the stress-strain curves of soils have an initial linear-elastic portion. This can be used to determine the initial tangent modulus, or elastic modulus. There is, however, some difficulty in using this method for these ballast triaxial tests. This is because most of the tests do not have a linear portion that is easy to identify, particularly at lower confining pressures, as seen in the example curves given in Figure 27. This method is therefore open to interpretation and unreliable. As an alternative, the 50% strength modulus is used. This is a secant modulus taken from the origin to the stress-strain curve at the point where 50% of the ultimate strength is reached. This method allows for more consistent and accurate measurement of the modulus, the results of which are presented in Table 6. It can be seen here that the angular Connecticut granite has modulus values that are often twice that of the naturally abraded ballast with natural fouling. Generally, for these two sets of tests, the modulus increases with confining pressure and with fouling content.
Table 6: 50% secant modulus for all triaxial tests.

<table>
<thead>
<tr>
<th>Water Condition</th>
<th>Confining Pressure</th>
<th>Secant Modulus at 50% Ultimate Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>0% Fouled</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Conn. Granite</td>
</tr>
<tr>
<td>Dry</td>
<td>5 psi</td>
<td>33.7</td>
</tr>
<tr>
<td></td>
<td>10 psi</td>
<td>39.4</td>
</tr>
<tr>
<td></td>
<td>15 psi</td>
<td>57.4</td>
</tr>
<tr>
<td>50% Field Capacity</td>
<td>5 psi</td>
<td>22.5</td>
</tr>
<tr>
<td></td>
<td>10 psi</td>
<td>53.9</td>
</tr>
<tr>
<td></td>
<td>15 psi</td>
<td>43.8</td>
</tr>
<tr>
<td>Field Capacity</td>
<td>5 psi</td>
<td>26.2</td>
</tr>
<tr>
<td></td>
<td>10 psi</td>
<td>31.6</td>
</tr>
<tr>
<td></td>
<td>15 psi</td>
<td>43.8</td>
</tr>
</tbody>
</table>

The Prestige mix fouling introduces some differences in behavior that can be looked at more closely. The modulus results from the natural fouling and prestige mix fouling tests with the abraded ballast are plotted against fouling content in Figure 33. It can be seen in the first pair of plots that in the dry condition, the modulus generally increases with increasing fouling content for both fouling types. With the addition of water, the tests typically show a drop in modulus from the clean to moderately fouled condition, followed by a slight increase in the heavily fouled condition. For a given fouling content, the modulus generally increases with confining pressure and decreases with increasing moisture content. The modulus of the tests with Prestige mix fouling is generally lower than their natural fouling counterparts. The decrease in modulus also occurs at a higher rate for the Prestige mix fouled tests when water is present.
Figure 33: Effects of fouling percentage on 50% ultimate strength secant modulus in different moisture conditions.
CHAPTER 5
EVALUATION OF BALLAST PERFORMANCE
UNDER HEAVY AXLE LOAD

Modified box tests were performed on reconstituted abraded ballast samples with both natural and clay mixture fouling to evaluate track settlement in different conditions. The box is made of half inch steel plate with internal dimensions of 4.3 ft (1.32 m) x 2.75 ft (0.82 m) x 1.125 ft (0.34 m). Wooden boards are attached directly to the upper portion of the box with bolts to increase the height by an additional 6 inches (15.42 cm). All the seams, joints, and bolt holes are sealed with silicone caulk to ensure that the box remains water tight. There is a drainage outlet at the bottom of the box to allow for the addition of water as well as discharge. These tests were performed to match the testing parameters of the previous work by Kashani [40] so that direct comparisons can be drawn between the results.

5.1 Specimen Preparation

Three specimens are prepared for the abraded ballast with natural fouling. Like the triaxial tests, fouling conditions of 0%, 15%, and 30% are used. Because the same ballast is used with the clay mixture fouling, an additional clean test was not needed, so only the equivalent 15% and 30% fouling specimens are prepared.

Each specimen is prepared in density-controlled lifts. The first three lifts are 3 inches (7.62 cm) followed by an additional three lifts of 2.5 inches (6.35 cm). As with the triaxial tests, when the lifts are prepared, the ballast is added first, followed by the required amount of fouling. The layer is tamped to promote mixing and achieve the appropriate bulk density. The first six layers provide a ballast depth of 16 inches. A quarter length of concrete tie is centered on top of this material. The rail section is placed atop the tie and a small load is applied to ensure that the rail is level and
properly seated in the ballast. A final 4-inch (10.2-cm) layer of ballast and fouling is placed around the tie, completing the specimen. An LVDT is oriented vertically over the tie. Because of flexure in the self-reacting frame, the LVDT is held in place by a metal frame which is attached directly to the box. This ensures accurate measurement of settlement alone, and not frame movement. Finally, a plastic sheet is placed over the material in the box to prevent loss of water from the specimen over the duration of testing. Figure 34 shows the completed test setup.

Figure 34: Completed box test setup showing LVDT placement and plastic covering to prevent moisture loss

5.2 Testing Procedure

Each box specimen is built in a self-reacting load frame which is equipped with a 110 kip (500 kN) MTS hydraulic actuator. Cyclic loading is applied sinusoidally at a frequency of 1 Hz. A minimum seating load of 0.2 kips (0.9 kN) is applied to prevent the actuator from lifting off the rail, which would create impact loads. The maximum load applied in each cycle is 15.6 kips (69.4 kN) which is equivalent to a static load of 39.4 tons (350.5 kN) and a dynamic axle load of 60 tons (533.8 kN).
The loading was determined using GEOTRACK, a three-dimensional model which accounts for substructure stress-dependent properties to determine track response [9]. This modeling was done by Kashani [40].

Each test starts in the dry phase and the moisture content is increased in steps. Each test has five different water conditions in which it is tested: dry, 1/3 field capacity, 2/3 field capacity, field capacity, and saturated. After reaching the appropriate number of loading cycles, the test is paused and water is added. For the 1/3 and 2/3 field capacity phases, the water is added from the top of the specimen and allowed to sit for 16 hours so that the water has time to fully penetrate the sample. Field capacity is achieved by filling the box from the bottom so that the material is completely saturated, and then immediately draining the excess water not held by the ballast and fouling. Saturation is achieved by completely filling the box with water and preventing water from flowing out of the bottom of the box.

The dynamic loading of rail typically does not exceed 10-20% of the static load, but higher peak loads can be caused by defects in the wheel or rail [58]. To simulate this, higher loads are applied in the saturated phase. After applying sufficient cycles of the 60-ton (533.8-kN) equivalent load to the saturated specimen, the test is stopped, and additional saturated phases are performed with 25%, 50%, and 75% increased load factors. The maximum loads applied are 19.5, 23.4, and 27.3 kips (86.7, 104.1, 121.4 kN), which are equivalent to static axle loads of 49.3, 59.1, and 69.0 tons (439, 526, and 614 kN) which gives dynamic axle loads of 75, 90, and 105 tons (667, 801, and 934 kN).

Table 7 provides the full loading schedule of each test along with equivalent load and MGT for each test. The MGT is based on the equivalent static axle loads. Note that the clean box test does not have 1/3 and 2/3 field capacity phases. This is because clean ballast has such a low field capacity that there is no practical difference between the intermittent water contents, so they are not used. Similarly, the clean
test is not subjected to higher loading when saturated. Without the presence of fouling, contact between the ballast particles is maintained, even when saturated, so the presence of water has less effect on the ballast performance. This also saves significant time when performing tests.

Table 7: Loading schedule for box tests.

<table>
<thead>
<tr>
<th>Fouling Condition</th>
<th>Water Content Phase</th>
<th>Load Applied (kips)</th>
<th>Equivalent Axle Load (tons)</th>
<th>Cycles in Phase</th>
<th>MGT Cum.</th>
</tr>
</thead>
<tbody>
<tr>
<td>0%</td>
<td>Dry</td>
<td>15.6</td>
<td>39.4</td>
<td>121,900</td>
<td>9.6</td>
</tr>
<tr>
<td></td>
<td>FC</td>
<td>15.6</td>
<td>39.4</td>
<td>225,500</td>
<td>17.8</td>
</tr>
<tr>
<td></td>
<td>Saturated</td>
<td>15.6</td>
<td>39.4</td>
<td>200,000</td>
<td>15.8</td>
</tr>
<tr>
<td>15%</td>
<td>Dry</td>
<td>15.6</td>
<td>39.4</td>
<td>200,000</td>
<td>15.8</td>
</tr>
<tr>
<td></td>
<td>1/3 FC</td>
<td>15.6</td>
<td>39.4</td>
<td>202,000</td>
<td>15.9</td>
</tr>
<tr>
<td></td>
<td>2/3 FC</td>
<td>15.6</td>
<td>39.4</td>
<td>204,500</td>
<td>16.1</td>
</tr>
<tr>
<td></td>
<td>FC</td>
<td>15.6</td>
<td>39.4</td>
<td>427,500</td>
<td>33.7</td>
</tr>
<tr>
<td></td>
<td>Saturated</td>
<td>19.5</td>
<td>49.3</td>
<td>255,000</td>
<td>25.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>23.4</td>
<td>59.1</td>
<td>501,000</td>
<td>59.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>27.3</td>
<td>69.0</td>
<td>327,000</td>
<td>45.1</td>
</tr>
<tr>
<td>30%</td>
<td>Dry</td>
<td>15.6</td>
<td>39.4</td>
<td>200,000</td>
<td>15.8</td>
</tr>
<tr>
<td></td>
<td>1/3 FC</td>
<td>15.6</td>
<td>39.4</td>
<td>200,000</td>
<td>15.8</td>
</tr>
<tr>
<td></td>
<td>2/3 FC</td>
<td>15.6</td>
<td>39.4</td>
<td>200,000</td>
<td>15.8</td>
</tr>
<tr>
<td></td>
<td>FC</td>
<td>15.6</td>
<td>39.4</td>
<td>415,000</td>
<td>32.7</td>
</tr>
<tr>
<td></td>
<td>Saturated</td>
<td>19.5</td>
<td>49.3</td>
<td>283,300</td>
<td>27.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>23.4</td>
<td>59.1</td>
<td>500,200</td>
<td>59.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>27.3</td>
<td>69.0</td>
<td>350,350</td>
<td>48.3</td>
</tr>
</tbody>
</table>

Like the triaxial tests, the box test data is collected using a DATAQ Instruments DI-710 data acquisition box and WinDAQ software. The LVDT data is collected and processed to determine elastic and total settlements of the tie and, by extension, the ballast. These data, along with load data from the actuator, can also be used to determine rate of settlement and test modulus.

5.3 Box Test Results

5.3.1 Plastic, Elastic, and Total Ballast Settlement

The cumulative settlement curves for the clean box tests, shown in Figure 35, provide a good baseline of the ballast settlement behavior. The previously tested granite
experiences an initial settlement at the beginning of each water content phase, with the settlement curve mostly flattening out after approximately 10,000 cycles. The abraded ballast has similar higher initial settlement, but the rate of settlement does not slow to the same extent as the granite. This is particularly pronounced in the dry phase, which has plastic settlements nearly 4 times as large as the granite at the end of the phase. The larger settlement from the abraded ballast may be due to increased rearrangement of particles facilitated by the more rounded particle shapes.

Figure 35: Plastic and total cumulative settlement curves for angular ballast and naturally abraded ballast in clean box tests.

Another thing to note from these results is the elastic settlement, which is represented by the difference in the plastic and total settlement. The elastic settlement is consistent throughout the duration of the test. The granite ballast has an average elastic settlement of 0.02 inches (0.5 mm) while the abraded ballast has a higher elastic settlement of 0.07 inches (1.8 mm). This difference is further reflected by the higher test moduli of the granite ballast box tests versus the abraded ballast box tests, as seen in the first plot in Figure 36. It can also be seen that the modulus is
approximately the same for the ballasts regardless of fouling condition. Since these results remain consistent, it suggests that the ballast material itself is most influential in determining elastic settlement and test modulus, while the moisture condition, fouling type, and fouling content have very little influence on these parameters. These results further highlight the importance of selecting suitable rock for ballast.

The introduction of fouling to the ballast leads to some changes in the ballast behavior. Figures 37 and 38 shows the results of the 15% fouled and 30% fouled box tests, respectively. Looking first at the 15% tests, it is observed that the addition of fouling increases the overall settlement of both ballasts. The abraded ballast again has higher settlements than the granite ballast, particularly in the dry and partially saturated phases. The addition of clay into the fouling mixture leads to more variable settlement behavior. In the phases with water contents less than field capacity, the Prestige mix fouled ballast settles less than the natural fouling alone. By contrast, during the field capacity phase, the Prestige mix fouled ballast underwent a settlement more than double that of the naturally fouled ballast. Interestingly, the settlement curves in the saturated phases, including those with increased loading, are very similar.

Normally, the presence of clay fines in fouling is regarded as particularly problematic to ballast [58], but the 15% box test doesn’t show higher settlement than the natural fouling test. One possible explanation for this behavioral shift in the ballast-fouling mixture is the behavior of the Prestige clay itself. Prestige has a very low unit weight and is not easily compacted in the dry condition. Because it is a kaolin, the clay contracts with the addition of water. This contraction may have actually increased contact between ballast particles during the 1/3 and 2/3 field capacity phases, increasing the overall strength of the material, resulting in the lower settlements. Once field capacity is reached, a larger settlement occurs, which may indicate that the clay had fully contracted by this point. It should be noted that this
Figure 36: Test modulus measured for all box tests, with a smoothing function applied for clarity.

Explanation is based solely on experience working with the material and assumptions drawn about the behavior of the material. Confirming this hypothesis would require further research to confirm.
Figure 37: Plastic and total cumulative settlement curves for 15% fouled box tests. Results from previous tests on granite ballast and current tests on abraded ballast with natural fouling and Prestige clay fouling mixture are included.

Figure 38: Plastic and total cumulative settlement curves for 30% fouled box tests. Results from previous tests on granite ballast and current tests on abraded ballast with natural fouling and Prestige clay fouling mixture are included.
Despite some differences in behavior, the 15% fouled settlement curves are overall very similar regardless of the ballast type or fouling material. However, the 30% fouled tests show different behaviors from one another. The granite ballast settlement in the undersaturated phases is not much larger than the same material with 15% fouling. However, once saturated, settlements occur at much higher rates, which continues to increase with higher loading. The naturally abraded ballast test with 30% natural fouling settles more than twice as much as the granite in the dry phase. Settlements are again higher, but more moderate, in the 1/3 and 2/3 field capacity phases. In the field capacity phase, settlement of the natural ballast is more than 6 times larger than the granite. During the saturated phase, somewhere between 2.8 and 4.4 MGT of loading, the natural ballast experiences a basal failure made evident by a large settlement as well of a rotation of the tie, which is pictured in Figure 39. Together, this evidence indicates a complete loss of support below the tie.

This test marks the first known failure of ballast in a box test, but it does not indicate that such a failure would occur in a real track. In the test, the tie has no fixity and its movement is restrained only by the surrounding ballast. In actual track, the rail is secured to each tie, adding a fixity which prevents rotation of individual ties. While the box failure shows that the fouled ballast is a problematic material in the track, it should not be interpreted as evidence of true track failure.

The 30% fouled test with the Prestige clay fouling mixture has initial settlements lower than the granite ballast in the early phases, similar to the 15% fouled tests. The amount of settlement increases moderately at 2/3 field capacity, but there is a drastic increase upon reaching field capacity. Like the abraded ballast with natural fouling, the overall settlement in this phase is 6 times higher than the granite. However, it can be seen that the settlement with the Prestige mix fouling begins to level off. This settlement rate continues into the saturated phase.

Both of the tests with Prestige mix fouling ultimately result in lateral failures.
The 15% test fails after increasing the loading to the equivalent axle load of 90 tons (801 kN), which caused a sliding of the tie, as well as movement of the entire box within the frame. The 30% test fails much later in its test, when the 105-ton (934-kN) equivalent axle load is reached. One of the notable aspects of these failures is the lack of indicators to predict them. The 30% fouled test with natural fouling experienced increased settlements beyond what might be expected, which would indicate an imminent box failure. The tests with Prestige mix fouling did not have these warning signs. These sliding failures in the box are similar to the basal failure in that they are unlikely to occur in real track. Since each tie is fastened to the rail, lateral sliding of a single tie would be prevented by restraint provided by adjacent ties.

The results between the 15% and 30% tests can also be compared directly for a better understanding of the effects of fouling. Figure 40 provides the cumulative plastic settlements from all of the 15% and 30% fouled box tests. Additional plots that compare settlement rates at different water contents and fouling conditions are
provided in Appendix B for comparison. The angular granite ballast exhibits similar settlement rates in the undersaturated phases of the test. Once saturation is reached, increased fouling increases the settlement rates by more than double. The abraded ballast with natural fouling sees immediate negative effects from the addition of fouling. The settlement in the dry phase for this material more than doubles when fouling is increased from 15% to 30%. In the field capacity phase, the settlement is more than 5 times greater with this increase in fouling. Increased settlement rates persist throughout the 30% fouled test until failure is reached in the saturated phase. The abraded ballast with Prestige mix fouling has lower settlement rates in the dry and 1/3 field capacity phases when the fouling is increased from 15% to 30%. During the 2/3 field capacity and field capacity phases, the settlement rate is much higher for the 30% test. However, the settlement rate slows in the field capacity phase, and this rate continues through the saturated phases. The rate of settlement is only slightly larger than the comparable 15% test.

Figure 40: Cumulative plastic settlement measurements for all 15% and 30% fouled box tests.
Based on the box results from this study and the previous work, it can be seen that the addition of fouling material increased overall settlements for each ballast-fouling combination. At the 15% fouled condition, the largely similar behavior between the three tests seems to indicate that the ballast particles still dominates the overall behavior of the specimen. Once fouling is increased to 30%, however, the behavior of the three tests varies more, indicating that the different fouling materials are beginning to have a larger effect on the behavior of each specimen. It is likely that further increases to fouling would result in more variation in the settlement behavior.

It has been argued that there is a fouling content threshold at which behavior of ballast completely changes, but no single ratio has been used to define it, due to the differing nature of soils [16]. The results here seem to suggest that this change may begin at 30% fouling. This would need to be confirmed with further testing of different fouling contents and more ballast and fouling materials. Better mixing models may provide more insight to determine at which fouling content the fouling material begins to dominate ballast behavior. Additionally, different methods of measuring fouling, such as PVC, may prove to be more useful when defining this threshold.

5.3.2 Failure Mechanisms in the Box Test

Two different failure mechanisms occurred during the course of box testing in Phase II of this project. The failure that occurred in the 30% fouled test with abraded ballast and natural fouling was fairly easy to identify. During the field capacity phase, the ballast settled steadily and at a high rate of approximately 0.02 in/MGT. Once saturation is reached, a large settlement and tie rotation indicate a failure. This can be readily identified as a basal failure, not dissimilar to a bearing capacity failure of a shallow foundation footing. It should be noted here that such a failure is unlikely to occur in real track. The rotation that occurred in this case would be prevented by the rail being secured to adjacent ties.
The failure mechanism occurring in the 15% and 30% fouled tests with the Prestige fouling mixture is more difficult to identify. Observationally, it was noted that the tie was undergoing slow rotations and potentially some lateral sliding, particularly after field capacity and saturation were reached. These movements were made evident largely by use of spirit levels applied to the surface of the tie. It is also observed that the center line of the rail and the actuator begin to deviate. In both cases, failure occurred once the center lines deviated enough that the actuator rotates around the top of the rail, causing a lateral sliding of the tie. However, the rotation and sliding did not appear to be progressive, and there was no major settlements occurring, so predicting when such a behavior could occur may be difficult based on these results.

These box test failures are not necessarily indicative of failures that would occur in the track. These tests only use a single quarter-length of tie with no fixity. Therefore it is not restrained in any way other than by the surrounding ballast. The restraint provided by the rails in actual track would prevent these failures from happening.
CHAPTER 6
RELATIVE DENSITY OF BALLAST

Commonly, ballast is placed at the maximum density possible to achieve higher strength and track stability, but there are no criteria for what this density is. Little work has been done to determine what the possible range of density is for ballast, or how fouling may affect that range. Relative density of a soil can greatly influence its engineering behavior [26] so it is important for us to better understand the range of densities possible in ballast and fouled ballast.

6.1 Specimen Preparation

Minimum and maximum density tests were performed on the granite ballast with granite stone dust fouling and on the abraded ballast with natural fouling. Tests were performed on clean ballast and on mixtures of 15%, 30%, 60% and 80% fouled ballast. Tests were also performed on pure fouling. The different ballast-fouling mixtures were prepared by measuring the appropriate mass of ballast and fouling, and placing both in a large tray. The materials were then manually mixed thoroughly for sufficient time to ensure homogeneity of the sample.

6.2 Testing Procedure

There is no found study which has used the ASTM standard procedures for minimum and maximum density to determine the index densities of ballast. These tests are most commonly performed on sand-size soils, which requires a standard mold with volume 0.1 $ft^3$ (2830 $cm^3$). However, the ballast particles are too large for this size mold. The standards do accommodate particles up to three inches in diameter, but this requires a larger mold with a volume of 0.5 $ft^3$ (14200 $cm^3$). Because it is less common, this mold is not available at the University of Massachusetts Amherst, and
purchasing a standard mold is cost prohibitive. An alternative that is equal in volume or larger must be used so that size effects are not an issue.

A large consolidation mold used for ballast was in storage in the laboratory. This mold has a diameter of 12 inches (30.5 cm), a height of 12 inches (30.5 cm), and a volume of 0.78 ft$^3$ (22100 cm$^3$) and was used for this testing. The mold was calibrated by filling it with water to determine the volume, as outlined by ASTM. The minimum density tests were first performed in general accordance with Method B found in ASTM D4254-14 [5]. Method B determines minimum density by “depositing material into a mold by extracting a soil filled tube.” According to the standard, this method should be used only on samples with 100% of materials passing the 3/4” sieve. However, the alternative method for ballast-size particles requires use of a large funnel or placement by hand using a scoop. It was decided that these methods would introduce too much error and that given an appropriately size tube, Method B would provide more consistent results. The tube used should have an inside diameter around 0.7 times the inside diameter of the mold and accommodate 1.25 to 1.3 times the volume of the mold. Therefore an 8-inch (20.3-cm) diameter mold is used. The tube is placed inside the mold and filled using a scoop. The tube is quickly lifted, allowing the sample material to overfill the mold. Normally, at this point, the top of the sample is trimmed to be flush with the top of the mold, but this cannot be done with ballast sized particles. Instead, the particles are removed by hand in such a way that the particles extending above the top of the mold are about equal to the larger voids just below the top of the mold, as described by the standard. An example of the overfilled mold and the mold after manual removal of pieces to level the sample can be seen in Figure 41. Each specimen is weighed at this point, and the density is determined.

The maximum density tests were performed in general accordance with Method 1A found in ASTM D4253-14 [5]. This standard uses the same mold as the minimum
density test. The mold is filled and loaded with a surcharge weight, then shaken on a vibratory table. Modifications were made to the consolidation mold and the vibrating table so that they could be securely attached. The surcharge weight assists with the densification of the sample when shaken. The standard calls for the surcharge weight to provide a surcharge stress of \( 2 \pm 0.02 \text{ lb/in}^2 \) (13.8 ± 0.1 kPa). For the mold in use, a custom made 265-lb (120-kg) surcharge would be required. To serve as the surcharge, a 1.5-in (3.8-cm) thick, 12-inch (30.5 cm) square steel plate was milled to fit inside the mold. A 2-in (5.1-cm) diameter steel bar was welded to the center of the plate perpendicularly. Eight Olympic style weights of 25 lbs (11.3 kg) each were stacked on the bar and secured with a clamp. The mold was filled, the surcharge applied, and the table vibrated according to the standard. Figure 42 shows the completed maximum density test setup. After the vibration was completed, the weights were carefully removed. Depth measurements were taken from the top of the
mold to the top of the surcharge plate. The average of six measurements was used to determine the material volume. The plate and bar are then removed, and the mold is detached from the table, and the sample and mold are weighed to determine the mass, and the data are used to calculate the density of that material in the mold.

Figure 42: The density mold set up for maximum density test, secured to shaker table and with surcharge weight applied.

6.3 Results

Five minimum density tests were performed for each ballast-fouling mixture. While Method B was used because of the consistency of results, the rate of lifting the tube and the process of leveling material to the top of the mold still introduces some human influence on the results. The results of the multiple tests are averaged in an attempt to reduce any error from user variability. Only one maximum density test is performed on each ballast-fouling mixture since this is a mechanically controlled
process in which human error is already minimized. The density, void ratio, and unit weight of the soil matrix is determined for each test and plotted.

Figure 43 shows the results from all of the minimum and maximum density tests presented as unit weights, which is simply the density multiplied by acceleration due to gravity. The same data are presented in Figure 44 as void ratios. Note that the minimum density tests result in the maximum void ratio, and therefore are shown in the upper curve on these plots. Likewise, the maximum density tests give the minimum void ratio and these data are presented in the lower curve on each plot.

These data reveal important characteristics of the ballast-fouling packing structure which require some explanation. It can be seen that for both ballasts, as the fouling content increases, there is first an increase in unit weight, followed by a decrease. Because the fouling is a finer grain size than the ballast, it first fills in the voids of the ballast. Better material packing is achieved, resulting in higher density. At some fouling level, all of the voids in the ballast will become filled, and the maximum packing is achieved. This would be represented by the peak of the curve where the two linear portions intersect. It can also be seen that the range of densities becomes narrower as this point is approached, showing that more efficient packing is achieved. As fouling increases from this point, the fouling itself makes up the majority of the matrix and the ballast particles begin to separate and lose contact with one another, resulting in a drop in density. At the highest fouling levels, the ballast can be thought of as inclusions in a fouling matrix.

These data show that around 40% fouling is the point at which the fouling begins to dominate the density behavior of the ballast-fouling mixture. This ratio is only indicative of a change in the packing structure, and does not necessarily translate to a change in strength or volumetric behavior in the ballast at this point.

The density behavior displayed by the ballast-fouling mixtures is very similar to that of the silt-sand mixtures investigated by Chang et al [10] [11] [12]. The same
Figure 43: Results from all density tests for both (a) abraded ballast and (b) granite ballast, presented as unit weight. A peaked curve can be seen by referencing Figure 18 in the literature review chapter, which presents density versus fines content for steel shots. That work uses past data on the void ratio of sand-silt mixtures and creates a model to predict the void ratio...
Figure 44: Results from all density tests for both (a) abraded ballast and (b) granite ballast, presented as void ratio.

of any such mixture. With appropriate accompanying testing, these data can also predict one dimensional settlement of the different mixtures using the model. It is intended to adopt the model using the results from these ballast-fouling mixture tests
so that similar predictions can be made for this material. This could help industry better understand the relationship between fouling content and track settlement.
CHAPTER 7
CONCLUSIONS AND FUTURE WORK

7.1 Conclusions

This dissertation presents the results from a series of consolidated drained triaxial tests and modified box tests which investigate the strength properties and deformation characteristics of railroad ballast in different fouling and moisture conditions. These tests use abraded ballast and two different fouling types, the results of which are compared to previous testing which used angular ballast. Additionally, a series of minimum and maximum density tests which investigate the packing structure of ballast-fouling mixtures are presented. The primary results from these investigations are as follows:

- The Connecticut granite and abraded ballast achieved similar strengths and friction angles to one another in most fouling and water content conditions, showing little correlation between angularity and strength. This is in agreement with studies by Vallerga et al. [66] and Holtz and Gibbs [27], but is in disagreement with studies by Indraratna et al. [35] [37], Koerner [44], and Holubec and D’Appolonia [28].

- The abraded ballast with the Prestige mix fouling had lower strengths and friction angles than the abraded ballast with natural fouling alone, showing the negative effects of plastic fines in fouling materials. This supports earlier work by Indraratna et al. [36] which used pure clay fouling.

- Abraded ballast is much more susceptible to deformations than angular ballast, with generally higher initial contractions occurring regardless of fouling or moisture condition.

- The addition of plastic fines to the fouling makes the ballast behavior less pre-
dictable, with much wider variation in the volumetric strain curves.

- Use of a clay mix fouling revealed the inadequacy of the percentage fouling measure of quantifying fouling present in the ballast. This study used percent void contamination (PVC) as an alternative, but other options such as void contamination index (VCI) could also be considered.

- Settlements in the box test complement triaxial test results, with higher initial contractions shown by the abraded ballast in most phases.

- The elastic settlement of the box tests remained consistent throughout testing, regardless of the fouling type, quantity, water content, or amount of loading. There was, however, a difference in elastic settlement between the angular and abraded ballast. This suggests that the elastic settlement of track is dependent on the ballast alone, with little influence from other factors.

- The first known basal failure in a box test occurred on the 30% fouled test with abraded ballast and natural fouling. This suggests that the angular ballast from previous testing is more capable of accommodating larger quantities of fouling and maintaining track support. This supports conclusions in a study by Han [22].

- The two box tests with abraded ballast and Prestige mixture fouling both experienced lateral failures. During these tests, gradual sliding and rotations of the tie were observed, indicating a different response to loading that will likely require additional testing to better understand.

- The box test settlement curves reveal similar behavior between the ballasts at 15% fouling, while the 30% fouling tests vary in settlement behavior. This suggests that the fouling begins to have a greater influence on the ballast behavior in the 30% fouled condition.
While the industry often focuses on fouling quantity, these results show that particle angularity and the type of fouling material are both important considerations for track maintenance. Abraded ballast may be serviceable in track, but its higher susceptibility to deformations can be problematic, particularly at higher fouling contents. Fouling containing plastic fines has the potential to further increase deformations and variability of behavior, as well as change the failure modes of ballast. This research shows that when assessing the suitability of the railway substructure, approaching the situation with these additional factors may be more suitable.

7.2 Contribution to Geotechnical Profession

The results of this study can be used in railway geotechnics in the following ways:

- The extensive triaxial and box tests data in different fouling and moisture conditions can help better understand risk factors in track.

- A better understanding of the influence of particle angularity on ballast performance can lead to more refined standards for ballast renewal efforts.

- The detailed negative influence of plastic fines present in fouling can help pinpoint problematic areas in the track, leading to more targeted maintenance.

- The shown inadequacies of the percentage fouling method should provide further support for adopting and standardizing alternative methods of quantifying fouling.

- The basal failure showed a noted lack of reduction in settlement rate leading up to the event, providing a framework to better identify the locations of more problematic materials in the track.

- Relative density testing can provide methods for better determining if ballast has been placed at an adequate density for traffic loading.
• Density testing a ballast-fouling mixtures can be used toward developing mixing models which will help to better understand the packing structure of material in the track.

7.3 Proposed Future Work

The laboratory tests in this research program have provided good insight into the effects of ballast abrasion and fouling type on the performance of railroad ballast. An important discovery in this testing was the potential inadequacy of the percentage fouling measure for quantifying fouling. The work on the relative density of fouled ballast also provides a good starting point for better understanding the density state of ballast. All of this work raises additional questions that could be addressed through some of these suggestions for future research:

• Perform triaxial tests in the saturated condition on these materials to correlate results to box tests.

• Perform additional tests at fouling percentages around 30% fouled to better understand the point at which fouling begins to more greatly influence ballast behavior.

• Begin quantifying fouling using the percent void contamination method over percentage fouling and use this as the basis for comparing tests.

• Use light weight deflectometer (LWD) on box test to measure modulus of the ballast in different conditions and relate to the moduli measured in the triaxial tests.

• Perform additional maximum and minimum density tests using more ballast types and gradations to better assess the range of possible densities under different fouling conditions and fouling types.
• Apply minimum and maximum density results to the mixture model developed by Chang et al. [12] to adapt for use with ballast-fouling mixtures

• Perform 1-D consolidation tests on ballast-fouling mixtures. Apply these results to the ballast-fouling mixture model to better predict track settlement.
APPENDIX A
ADDITIONAL TRIAXIAL TEST TABLES AND FIGURES
Table 8: Friction angles and ultimate strengths for all triaxial tests.

<table>
<thead>
<tr>
<th>Water Condition</th>
<th>Confining Pressure</th>
<th>0% Fouling</th>
<th>15% Fouling</th>
<th>30% Fouling</th>
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<tr>
<td></td>
<td></td>
<td>Conn. Granite</td>
<td>Naturally Abraded</td>
<td>Conn. Granite</td>
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<td></td>
<td>psi</td>
<td>degrees</td>
<td>kPa</td>
<td>degrees</td>
</tr>
<tr>
<td>Dry</td>
<td>5</td>
<td>47.4</td>
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<tr>
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<td>10</td>
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<tr>
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<td>528</td>
<td>506</td>
<td>529</td>
</tr>
<tr>
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<td>47.7</td>
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<td>566</td>
<td>542</td>
</tr>
<tr>
<td>FC</td>
<td>5</td>
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<tr>
<td></td>
<td>10</td>
<td>166</td>
<td>366</td>
<td>434</td>
</tr>
</tbody>
</table>
Figure 45: Clean, dry, 5 psi confining
Figure 46: Clean, dry, 10 psi confining
Figure 47: Clean, dry, 15 psi confining
Figure 48: Clean, 50% field capacity, 5 psi confining
Figure 49: Clean, 50% field capacity, 10 psi confining
Figure 50: Clean, 50% field capacity, 15 psi confining
Figure 51: Clean, field capacity, 5 psi confining
Figure 52: Clean, field capacity, 10 psi confining
Figure 53: Clean, field capacity, 15 psi confining
Figure 54: 15% fouled, dry, 5 psi confining
Figure 55: 15% fouled, dry, 10 psi confining
Figure 56: 15% fouled, dry, 15 psi confining
Figure 57: 15% fouled, 50% field capacity, 5 psi confining
Figure 58: 15% fouled, 50% field capacity, 10 psi confining
Figure 59: 15% fouled, 50% field capacity, 15 psi confining
Figure 60: 15% fouled, field capacity, 5 psi confining
Figure 61: 15% fouled, field capacity, 10 psi confining
Figure 62: 15% fouled, field capacity, 15 psi confining
Figure 63: 30% fouled, dry, 5 psi confining
Figure 64: 30% fouled, dry, 10 psi confining
Figure 65: 30% fouled, dry, 15 psi confining
Figure 66: 30% fouled, 50% field capacity, 5 psi confining
Figure 67: 30% fouled, 50% field capacity, 10 psi confining
Figure 68: 30% fouled, 50% field capacity, 15 psi confining
Figure 69: 30% fouled, field capacity, 5 psi confining
Figure 70: 30% fouled, field capacity, 10 psi confining
Figure 71: 30% fouled, field capacity, 15 psi confining
APPENDIX B

BOX TEST SETTLEMENT AND SETTLEMENT RATE CURVES
Figure 72: Clean box, dry, 15.6 kips

Figure 73: Clean box, field capacity, 15.6 kips

Figure 74: Clean box, saturated, 15.6 kips
Figure 75: 15% fouled box, dry, 15.6 kips

Figure 76: 15% fouled box, 1/3 field capacity, 15.6 kips

Figure 77: 15% fouled box, 2/3 field capacity, 15.6 kips
Figure 78: 15% fouled box, field capacity, 15.6 kips

Figure 79: 15% fouled box, saturated, 15.6 kips

Figure 80: 15% fouled box, saturated, 19.5 kips
Figure 81: 15% fouled box, saturated, 23.4 kips

Figure 82: 15% fouled box, saturated, 27.3 kips
Figure 83: 30% fouled box, dry, 15.6 kips

Figure 84: 30% fouled box, 1/3 field capacity, 15.6 kips

Figure 85: 30% fouled box, 2/3 field capacity, 15.6 kips
Figure 86: 30% fouled box, field capacity, 15.6 kips

Figure 87: 30% fouled box, saturated, 15.6 kips

Figure 88: 30% fouled box, saturated, 19.5 kips
Figure 89: 30% fouled box, saturated, 23.4 kips

Figure 90: 30% fouled box, saturated, 27.3 kips
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