Chapter 3

Substructure

The track substructure includes the ballast, subballast, and subgrade layers that support the track superstructure of rails and ties. Track substructure behavior has a significant influence on track superstructure stability and performance as well as vehicle dynamics. The main function of the track substructure is to support the applied train loads uniformly and without permanent deformation that might affect the track geometry. This chapter focuses on the properties and roles of track substructure components of ballasted railway track structure, although other track structure types are briefly discussed.

3.1 BALLAST

Railway ballast is the crushed stone that forms the top layer of the substructure, in which the tie (sleeper) is embedded and supported. Mainline ballast material is usually large, uniformly graded crushed stone. Although crushed stone is used for a variety of engineering purposes, as railroad ballast it is subjected to a uniquely severe combination of loading stresses and environmental exposure. In particular, the upper portion of ballast that is directly below the tie is the zone that must endure the highest stresses from traffic loads and from the surfacing operation, leading to more rapid ballast deterioration in this area (Chrismer 1993). There are many quarries that supply crushed rock for other types of construction, but the rock’s suitability as ballast must consider the distinctive demands of the railroad environment. This should be kept in mind when evaluating new ballast suppliers, because good experience of other industries with a given source of rock may not translate into its success as railway track ballast.

This chapter’s treatment of ballast covers its material considerations, but the subject is also discussed in Chapter 4 with regard to ballast layer behavior and mechanics, and ballast life is discussed in Chapter 9.
3.1.1 Ballast functions

The acceptability of material for use as ballast must be judged by its ability to perform its intended functions. The ballast functions described by Hay (1982) and Selig and Waters (1994) include the following:

1. Resist applied loads (vertical, lateral, and longitudinal) to maintain track position.
2. Provide positive and rapid drainage of water.
3. Accommodate track surfacing and alignment maintenance.
4. Provide resilience needed to dissipate large and dynamic loads.
5. Provide adequate void space for storage of ballast-fouling material without interfering with ballast particle contact.

Additionally, Selig and Waters (1994) mention several secondary requirements for ballast: resist frost action, inhibit growth of vegetation, reduce propagation of airborne noise, and provide electrical resistance between rails.

The track cross section in Figure 3.1 shows the interface of the ballast and tie and several important ballast zones. As noted by Selig and Waters (1994), the ballast layer can be subdivided into these zones including the upper ballast layer, which is the zone where track maintenance disturbs the ballast and influences ballast performance, and the lower ballast, which is usually not disturbed by tamping maintenance. Additionally, this zone of upper ballast that supports the tie directly below the rail seat area is termed the tamping zone and is often the most heavily loaded ballast zone. The ballast below the crib should be noted as the subcrib ballast.

![Figure 3.1](image-url) (a,b) Ballast zones.

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of distinguishing between crib and subcrib zones arises from consideration of the zones affected by the applied train load and track maintenance, in particular, track surfacing. The tamper used in track surfacing lifts the track structure and has tines that are inserted in the ballast to force the rock particles under raised ties in the heavily loaded tamping zone.

### 3.1.2 Parent rock characterization

The type of parent rock that the ballast is crushed from will affect the derived ballast particle size, shape, and angularity of individual particles, which consequently affects the frictional resistance to sliding between two particles in contact. Some rock has cleavage planes and bedding layers that set up preferred failure planes that can control particle shape. An understanding of parent rock characteristics provides insight into ballast performance.

The characterization of crushed rock ballast begins with evaluating the strength and inherent structure of the parent rock mass including cleavage and bedding planes. The rock strength can be assessed by testing the unconfined compressive strength of an extracted rock core or by testing the ballast. Natural variations within a quarry should be evaluated to determine if they could adversely impact ballast shape, size, or performance. Once a potentially suitable parent rock mass has been identified, a detailed evaluation of the crushed rock aggregate should be undertaken. The following sections discuss some of the more common tests for ballast aggregates.

### 3.1.3 Aggregate characterization

The performance and longevity of ballast particles is dependent on the material properties and loading conditions. Aggregate characterization is important for judging its performance as ballast, whether one is determining suitable quarry sources for new ballast or is assessing the performance of existing ballast in track.

#### 3.1.3.1 Grain size distribution

The grain size distribution (gradation) of ballast is the most common specification for new ballast and a common technique for the assessment of worn ballast. The grain size distribution of the aggregate mass allows for reliable correlation to strength, deformation, and drainage characteristics. Mainline ballast specifications typically require a relatively narrow range of particle sizes, which maximizes interparticle void volume. This large void volume facilitates drainage and provides for substantial storage of ballast fouling material. When narrowly graded ballast is adequately compacted, it performs well and provides superb drainage and fouling material storage. More broadly graded ballast will provide increased strength and resistance.
to deformation due to the denser packing arrangement of the particles, but broadly graded ballast can be expected to have a lower void volume than narrowly graded ballast. Although a more broadly graded ballast can provide superior resistance to deformation and might provide the desired drainage and fouling material storage capacity, the final challenge then becomes material handling and transport because the particles will tend to segregate during transportation to the site. The as-placed ballast gradation is critical to ballast performance and often most difficult to assess. Keeping in mind sample size requirements from the American Society of Testing Materials (ASTM), it is also important to follow sampling guidelines to ensure that a representative sample is obtained when evaluating field placement.

The grain size distribution test (ASTM D6913) consists of placing a sample in the top of a stack of sieves arranged in the order of decreasing opening size from top to bottom (Figure 3.2). The particles in the top sieve falls through the stack as it is shaken, and particles are retained on the sieves through which they are too large to pass. The weight retained on each sieve is recorded and expressed as a percentage of the original weight. The percent passing each sieve size is typically plotted on a semilog chart of sieve size versus percent passing, which is the grain size distribution plot (Figure 3.3).

The allowable grain size range of two common ballast gradations, according to the American Railway Engineering and Maintenance-of-Way Association (AREMA) manual of recommended practices, is shown in Figure 3.3, and the percent passing certain sieve sizes for a few gradations are presented in Table 3.1.

Particle size is an important factor in ballast performance because the amount of void space between particles increases with particle size, and therefore the amount of void volume for storage of fouling fines increases.
However, the effect of particle size alone on strength is difficult to quantify because the associated effects of void ratio, particle shape, and texture, among other variables, also tend to affect the aggregate strength. This requirement for increased void volume must be balanced with the difficulty in working with and compacting larger particles. The compaction force required to densify ballast increases with increasing particle size.

As ballast deteriorates and becomes increasingly fouled, it is important to characterize the amount of fine-grained material to determine its expected performance and remaining life. The fine material that passes a #200 sieve is silt and clay sized, which is particularly damaging to the

Table 3.1 AREMA recommended ballast gradations 3, 4, and 24

<table>
<thead>
<tr>
<th>Sieve designation (ASTM E11-09)</th>
<th>Sieve opening size (mm)</th>
<th>AREMA ballast gradation Percent passing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>4</td>
</tr>
<tr>
<td>3”</td>
<td>75.0</td>
<td>–</td>
</tr>
<tr>
<td>2 1/2”</td>
<td>63.0</td>
<td>–</td>
</tr>
<tr>
<td>2”</td>
<td>50.0</td>
<td>100</td>
</tr>
<tr>
<td>1 1/2”</td>
<td>37.5</td>
<td>90–100</td>
</tr>
<tr>
<td>1”</td>
<td>25.0</td>
<td>20–55</td>
</tr>
<tr>
<td>3/4”</td>
<td>19.0</td>
<td>0–15</td>
</tr>
<tr>
<td>1/2”</td>
<td>12.5</td>
<td>–</td>
</tr>
<tr>
<td>3/8”</td>
<td>9.5</td>
<td>0–5</td>
</tr>
</tbody>
</table>
ballast permeability and strength in the presence of water. Dry sieving of a fouled ballast sample might result in incorrect assessment of the percentage of these fine particles if a portion of fine particles agglomerate to form a larger mass retained on the #200 sieve leading to the incorrect designation of this material as coarse grained. The presence of agglomerated fine-grained particles can be difficult to distinguish from small coarse-grained particles while conducting the test. If an incorrect assessment is made of the amount of fine particles contained in the ballast, it would cause incorrect inferences about future performance and ballast life. The wet sieve approach (ASTM C117) is very effective at separating the fine fraction by using water to disperse the finer particles so they pass through the openings of the #200 sieve.

Ballast deterioration tends to shift the grain size distribution to smaller sizes (to the right in Figure 3.3). To the extent that ballast is randomly crushed to a smaller size under traffic, it might seem that the grain size plot would shift fairly uniformly to smaller particle sizes with ballast degradation. However, it is more common for the grain size distribution of fouled ballast to develop a distinct “tail” in the distribution curve with perhaps 10%–20% or more particles finer than coarse sand indicating that ballast deterioration can be dominated by a wear mechanism that generates a predominance of small particles.

3.1.3.2 Shape, angularity, texture

Particle shape, angularity, and surface texture are important elements needed to assess potential strength characteristics of an assembly of particles. Shape, angularity, and surface texture are critical elements that affect ballast performance since they affect ballast interlocking, which layer contributes to ballast strength and deformation behavior. Well-proportioned particles that tend to be somewhat cubical are required for stable and strong ballast, whereas elongated, flaky, rounded, or smooth shapes should be avoided. Selig and Waters (1994) provide a detailed discussion of ballast particle shape, angularity, and surface texture requirements and assessment criteria.

Flaky or elongated particles have a specific dimension that is significantly greater than other dimensions, and these tend to set up preferential planes of weakness when present in large proportion and clustered together. Flat and elongated (F&E) ratio is defined as the ratio of the maximum dimension of an aggregate particle to the minimum one, and AREMA specifications currently allow a maximum of 5% by weight of total ballast having greater than 3 to 1 F&E ratio. An alignment of an abundance of flat or elongated ballast particles may lead to particle breakage, settlement, and possibly the development of slip failure surfaces and shear movement.

Surface texture contributes greatly to the interlocking of ballast particles that is required to develop a stable ballast layer. Particles with rough surface texture tend to interlock better than smooth. The roughness refers
to the surface texture of the particle with the most basic assessment of roughness being by touch where marble particles feel smooth and granite particles are rough. Surface texture is the main contributor to surface friction with rough particles having a higher friction factor and thus developing a higher friction force at the same normal force. An image showing the surface texture of several particles is illustrated in Figure 3.4.

Angularity is a measure of the sharpness of the edges and corners of an individual particle. Rounded particles tend to roll past one another under load, whereas angular particles interlock to resist the applied load. The angularity contributes to the ability of the particles to resist shear. A combination of both angularity and surface roughness are required for ballast particles to interlock properly and form a stable layer.

Assessment of both angularity and texture characterization typically rely on comparison with standardized charts where angularity is expressed on a qualitative rating scale of angular, subangular, subrounded, and rounded. Surface texture is described as rough, subrough, subsmooth, or smooth (see Figure 9.18 for ballast particle shape indices).

Improved techniques for measuring and analyzing ballast shape and roughness are available. Fractal analysis is an effective technique for quantifying both the angularity and the surface texture of particles (Hyslip and Vallejo 1997). In addition, Moaveni et al. (2013) describes an image-processing and analytical technique to better quantify particle shape. The number of fractured surfaces can also be used as an indicator of aggregate roughness (ASTM D5821) and is specified for a variety of coarse aggregate applications such as concrete and asphalt, including as a ballast specification (Selig and Waters 1994).

Particle shape, angularity, and surface texture most notably affect the mass shear strength of the ballast material, although it has been noted (Selig and Waters 1994) that increased angularity and roughness are related to increased ballast wear and breakage because the loaded contact point

Figure 3.4 Surface texture roughness observation from thin section.
between particles cause local stress concentrations and increased abrasion. However, the benefit of increased shear strength outweighs the disadvantage of increased breakage/wear potential, and therefore, it is desirable to have rough, angular ballast.

3.1.3.3 Petrographic analysis

Petrographic analysis is a visual technique to evaluate the source, composition, and nature of the material making up the sample and should be performed on representative samples to provide an indication of performance and weathering/breakdown-related potential of the material. Watters et al. (1987) note that the performance of ballast in the field depends on characteristics of the parent rock, which can be determined by petrographic analysis. Petrographic analysis can be a reliable aggregate quality test; however, the results are strongly dependent on the competence of the petrographer (Selig and Waters 1994). An investigation by Wnek et al. (2013) demonstrated the challenge in identifying small changes in petrographic characteristics that substantially influenced ballast performance. Petrographic examination involves visual assessment of specimens under microscopic examination including preparing “thin sections,” as depicted in Figure 3.5, among other tests. Thin sections are slices of rock polished to a thickness of 30 μm and examined under a polarizing microscope to determine mineral modes, grain size, grain morphology, and textural fabric.

Petrographic examination applies to both parent rock assessment and ballast performance assessment. When evaluating the parent rock material, petrographic analysis should be performed on the ballast material to evaluate the bulk composition, texture, and presence of secondary minerals.
that may affect performance. The petrographic analysis can indicate other potential mechanisms of weakness, as shown in Table 3.2.

### 3.1.3.4 Crushing and abrasion resistance

Crushing and abrasion of ballast are often leading causes of ballast degradation that can cause track settlement and drive track geometry maintenance costs. The tests considered in this section can be used to identify ballast that may be prone to crushing and abrasion and include Los Angeles Abrasion (LAA), Mill Abrasion (MA), bulk specific gravity, absorption, sulfate soundness, angularity, surface texture, and petrographic analysis. Whenever possible, standard testing methods such as the ASTM, British Standard (BS), or other equivalent standard specification methods should be used to ensure both the quality of results and the ability to readily duplicate the testing methods and compare results from other labs or ballast sources.

#### 3.1.3.4.1 Los Angeles Abrasion

The LAA test provides a measure of crushing resistance, which is used to evaluate ballast particle strength and characteristics to resist the breakage under the tie. The LAA test involves rotating 10 kg (22 lb) of ballast with 12 steel balls (5 kg total weight) for 1,000 revolutions in a steel drum (Figure 3.6). The rotation of the drum at 30–33 rpm results in impact between the steel balls and the ballast particles as they tumble in the cylinder, crushing the ballast. After 1,000 revolutions, the material is removed from the drum, and the sample is wash sieved on a 4.25 mm (#12) sieve. The LAA value is the amount of material passing the 4.25 mm sieve generated.
by the test as a percentage of the original sample weight. A small LAA value is desired for ballast as it indicates increased resistance to crushing. The version of the test most suited to ballast is ASTM C535, “Standard Test Method for Resistance to Degradation of Large-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine.” The LAA test is also designated as BS EN 1097-2 and BS EN 13450 Annex C.

BS 812-110 and 812-112 are other aggregate tests of the crushing resistance. In BS 812-110, a standard cup is filled with aggregate and a static load of 390 kN is gradually applied over a period of 10 min. The crushing value is taken as the percentage of fines produced during the test to the original sample weight. In BS 812-112, a standard cup is filled with aggregate, onto which a standard weight is dropped from a given distance. The aggregate impact value is the weight of smaller particles generated from impact as a percentage of the original sample weight. Taken together, BS 812-110 and BS 812-112 reasonably simulate the mechanisms of ballast crushing due to monotonically increasing load and of sudden impact loading. The standard test method is straightforward and reasonably fast, which are desirable characteristics. However, careful sample preparation is critical for accurate and reliable results.

Although the LAA and the BS-812 test conditions represent valid ballast degradation modes, an additional test is needed to measure ballast degradation due to abrasion by attrition through particle on particle grinding, which is a common ballast degradation mechanism.
3.1.3.4.2 Mill Abrasion

The MA test provides a measure of abrasion resistance of ballast and is an indicator of ballast hardness. The MA test concept is based on the micro-Deval abrasion test (ASTM D6928) for aggregate crushing and abrasion but has the advantage of testing separately for abrasion resistance without crushing, where crushing of particles is better measured in the LAA test. The MA is a wet abrasion test and consists of revolving 3 kg (6.6 lb) of material with a specified amount of water about the longitudinal axis of a 229 mm (9 in.) outside diameter porcelain jar (Figure 3.7) at 33 rpm for 10,000 revolutions. The rotation of the porcelain jar causes the ballast particles to tumble and roll over each other, resulting in wear without significant particle crushing as in the LAA test. The MA value is the amount of material passing the 0.075 mm (#200) sieve generated by the test as a percentage of the original sample weight. Low MA values are desired for ballast material. There is currently no ASTM standard for MA testing; however, Selig and Boucher (1990) discuss the test in detail and present recommended procedures.

Alternatively, ballast aggregate may be evaluated for similar abrasion characteristics with the micro-Deval abrasion test as described by ASTM D6928, “Standard Test Method for Resistance of Coarse Aggregate to Degradation by Abrasion in the Micro-Deval Apparatus” (alternatively specified in BS EN 1097-1). The micro-Deval abrasion test procedure results in some particle crushing that overlaps the LAA test specification. A better practice is to separate ballast particle wear and crushing characteristics in a ballast specification by using both the LAA value and the MA value separately (Klassen et al. 1987).
3.1.3.5** Bulk-specific gravity, absorption, sulfate soundness**

Bulk-specific gravity and absorption tests provide indicators of strength and breakage potential based on the concept that higher strength is associated with higher density aggregate. Bulk-specific gravity is typically determined using Archimedes principle based on water displacement. Absorption is an indication of the rock porosity that relates to its strength and freeze-thaw resistance. Absorption is defined as the weight of water absorbed by the particles divided by the dry particle weight, expressed as a percentage. Bulk-specific gravity and absorption tests are specified in ASTM C127, “Standard Test Method for Specific Gravity and Absorption of Coarse Aggregate.”

Sulfate soundness is a measure of a stone aggregate’s freeze-thaw resistance, which is important to assess where ballast is exposed to freezing temperatures and precipitation. The sulfate soundness test is a form of accelerated weathering of the particles and is conducted by subjecting stone aggregate repeatedly to saturated solutions of either sodium sulfate or magnesium sulfate. These solutions carry chemicals into the void space of aggregate particles, which then form crystals that exert internal expansion pressure to simulate the deterioration mechanism of freezing water. The weight of smaller particles generated from breakdown during the test is measured, and this weight is expressed as a percentage of the initial specimen weight. Sodium sulfate tests should be performed in accordance with ASTM C88, “Standard Test Method for Soundness of Aggregates by Use of Sodium or Magnesium Sulfate.” The results of the sulfate soundness test should be interpreted with an understanding of the inherent variability of these tests and the challenges associated with trying to simulate aggregate deterioration under freeze-thaw conditions. The sulfate soundness test results depend on the type of solution used during the test, and variability between labs using the sodium sulfate solution has been high leading to concern that the test may not be reliable (Selig and Waters 1994). The magnesium sulfate solution appears to provide less variability in results than the sodium sulfate solution (New York State DOT Geotechnical Test Method 21).

The sulfate soundness test using magnesium sulfate solution provides a reasonable indication of freeze-thaw variability. Other indications of freeze-thaw resistance, such as particle density and strength, must also be considered when evaluating results from sulfate soundness. When possible, field experience with the freeze-thaw resistance of ballast must be considered directly and weighed with laboratory test results.

3.1.3.6** Ballast aggregate specifications**

Table 3.3 presents specifications for the ballast aggregate characterization tests. Also included in this table are specifications recommended by AREMA (2013) and the Canadian Pacific Railroad (Klassen et al. 1987).
3.1.4 Ballast fouling

Ballast fouling is the process by which the voids between particles become filled with fouling material. The fouling material is derived from ballast deterioration from repeated loading and tamping and from sources external to the ballast. Although a variety of potential sources of fouling material are possible and fouling sources are often site-specific, the most common source is breakdown of the ballast itself. A definitive study by Selig and Waters (1994) reported that 76% of sites studied had ballast breakdown as the major source of fouling. The far less prevalent second largest source of fouling material, which was the predominant source of fouling at 13% of sites, was upward migration of material from a lower granular layer. In these cases, the lower layer was often old roadbed material that was mixed with the ballast at the interface of the layers. The old roadbed materials observed to foul ballast were typically granular rather than fine grained and did not infiltrate very far into the ballast. Ballast contamination from the subgrade is rare. The fact that fouled ballast often appears as mud leads to the common misconception that subgrade has pumped up into the ballast. The appearance of mud in the ballast typically results from the abrasion of ballast under traffic in the presence of water.

Although the most common source of fouling material is abraded or crushed ballast, other sources can include

1. Contaminants shipped with the ballast or material mixed with the ballast while it is handled or installed
2. Material dropped or spilled on the track from lading, or from traction sanding
3. Windblown material
4. Soil penetrating the ballast from below
5. Tie or other deteriorated track materials

Although any of these possible sources of fines could be the dominant contributor to fouled ballast at a specific site, it appears reasonable to assume that fouling material is generated from ballast breakdown unless there is
reason to believe that another cause is evident or likely. Color can be a clue to the fouling source, although color can often be misleading because the presence of only a small amount of fouling from certain single sources can control the color of fouling material that is predominately from other sources. For example, in locations where concrete tie wear is a problem, fouling material can take on a light gray color even though the abraded concrete generally makes up only a small percentage of the fouling material. On lines with significant coal traffic, coal dust from passing cars falls onto the track often giving a dark black color to the fouled material even if the coal fines are a minor fouling component. In locations where subgrade has penetrated the ballast, the fouling material may take on the color of the subgrade although the color of the subgrade may not be distinct. Ballast contaminants such as coal, subgrade, and other natural materials are often not distinct in color from other ballast contaminants, making visual clues unreliable. Any definitive assessment of the source of ballast fouling will need a test to assess the source and nature of the fouling material.

Ballast performance is much worse when fouling materials have plasticity compared to granular, nonplastic fouling material, making the characterization (plasticity, grain size, etc.) of the fouling material important for determination of track performance and maintenance requirements. ASTM D2488 Standard Practice for Description of Soils (Visual Manual Method) can provide clues about the plasticity of ballast-fouling material that can help to identify the likely source.

One measure of the amount of fouling material is the percentage of material passing the #4 sieve (opening size of 4.76 mm or smaller). Typical ballast gradation specifications limit the amount of material smaller than this size as delivered; therefore, the amount of material passing the #4 sieve is often related to the degree of fouling. A practical definition of fouling material is the presence of material that is of smaller particle size than is allowed according to the original specification for ballast. Selig and Waters (1994) define fouling material as the particles passing the #4 sieve, which is subdivided into coarse fouling, the particles between the #4 sieve and the #200 sieve, and fine fouling, the material passing the #200 sieve. Ballast performance has been observed to be significantly worse with fine-fouling material in the ballast compared to coarse-fouling material (Selig and Waters 1994).

Selig and Waters (1994) proposed the fouling index to quantify the degree of ballast fouling:

\[
FI \ (\text{fouling index}) = P_4 + P_{200}
\]

where:

- \( P_4 \) is the percent passing the #4 sieve (4.76 mm)
- \( P_{200} \) is the percent passing the #200 sieve (0.074 mm)
This accounts for the material passing the #200 sieve twice to accentuate the effect of the finer material due to its very large influence on permeability. The FI has been observed to correlate well with ballast performance problems.

As the ballast begins to become fouled, the initial fouling material generated is expected to be coarse particles due to breakage of asperities as the ballast compacts. Although the coarse broken particles may reduce ballast strength slightly, they are not expected to be very damaging to ballast performance as they fall into the voids between ballast particles and will not reduce ballast permeability appreciably. Coarse broken particles may even improve ballast performance slightly, if they inhibit ballast movement. This process, along with particle interlocking, is thought to be responsible for the more stiff response (similar to a more broadly graded material) of ballast after a significant number of load cycles because the coarse breakage may, under the right conditions, inhibit ballast particle movement by in effect wedging the ballast into position (Selig and Waters 1994; Indraratna and Salim 2006).

However, as load cycles accumulate, ballast wear continues and produces progressively smaller particles. As these finer particles accumulate, they will tend to inhibit ballast performance by retaining water and reducing drainage, creating the appearance of mud in the presence of water. The behavior of fouled ballast is further degraded if the fouling material behaves plastically (Han and Selig 1997). In general, fouling material in the ballast contributes to some degree of ballast drainage problems, settlement, and increased track maintenance.

The effect of fouled ballast on track performance is dramatic. Sussmann et al. (2001a) studied the effects of track condition and stiffness (Sussmann 2007) on track geometry profile deterioration rates and found that the geometry deteriorated most rapidly at sites where fouled ballast and drainage problems existed. A detailed geotechnical investigation of the track found that sites with poor drainage, fouled ballast, and subgrade failures had the most advanced deterioration. The fouled ballast sites had the highest rates of track geometry degradation when there was also a lack of drainage that retained water in the fouled ballast.

Efforts to develop a quick, reliable, and robust method to quantify the fouling condition of ballast relative to in-service performance have been pursued for decades. The fouling index by Selig and Waters (1994) provides a robust method of discriminating ballast conditions, but the heavy reliance on the percent passing the #200 sieve make this impractical to implement on a large scale due to the difficulty of reliably sieving material on such a very fine mesh in the field. Sussmann et al. (2012) developed a comparison of the fouling index to the criteria based on the amount of material passing the 3/4 in. sieve as developed by Canadian National (CN). Table 3.4 shows the comparative results where standard ballast gradations at varying fouling levels published in Selig and Waters (1994) were equated to CN criteria based on the percent passing the 3/4 in. sieve.
Notably, the CN comparison does not have corresponding data for clean to moderately clean ballast mainly because the criteria were established only for ballast nearing its end of life. Moderately clean ballast corresponds to less than 25% passing the 3/4 in. sieve. For the moderately fouled, fouled, and highly fouled conditions, a clear relationship between the breakpoints in the fouling index and the CN criteria were found. The benefit of the CN criteria is that the use of a single sieve with large openings lends itself to reasonably accurate testing in the field while reducing the need to transport samples to the laboratory for gradation analysis.

An alternative fouling index was developed by Spoornet (Vorster 2013) based on the percent passing \( (P) \) the 19, 6.7, 1.18, and 0.15 mm sieves:

\[
FI_{\text{Spoornet}} = 0.4P_{19} + 0.3P_{6.7} + 0.2P_{1.18} + 0.1P_{0.15} \tag{3.2}
\]

The use of four sieves provides a better indication of the deteriorated grain size distribution compared with that of new ballast and may be an improved indicator of ballast performance. However, this formulation for fouling index is moving away from the simplified form needed for field assessment. In addition, the small value of the 0.1 multiplier for the finest particles should be investigated because this appears counter to the argument that the finer material affects ballast performance most significantly.

Ballast cleaning intervention limits have been developed, and Table 3.5 compares several ballast-fouling intervention levels. Two main criteria are

<table>
<thead>
<tr>
<th>Agency or source</th>
<th>Criteria/size mm (in.)</th>
<th>Limit (%)</th>
<th>Rationale for intervention</th>
</tr>
</thead>
<tbody>
<tr>
<td>Selig and Waters</td>
<td>Fouling index (FI)</td>
<td>40</td>
<td>Highly fouled</td>
</tr>
<tr>
<td></td>
<td></td>
<td>30</td>
<td>Drainage reduction</td>
</tr>
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<td>FI_{\text{Spoornet}}</td>
<td>80</td>
<td>Gradation change</td>
</tr>
<tr>
<td>Canadian National (Ruel)</td>
<td>19 mm (3/4 in.)</td>
<td>40</td>
<td>Highly fouled</td>
</tr>
<tr>
<td>UIC</td>
<td>14 mm (1/2 in.)</td>
<td>30</td>
<td>N/A</td>
</tr>
<tr>
<td>ERI</td>
<td>22.4 mm (7/8 in.)</td>
<td>30</td>
<td>N/A</td>
</tr>
</tbody>
</table>
used: either a fouling index or single sieve size percent passing criteria. The fouling index approach requires laboratory testing to develop the required grain size distribution, while the single sieve approach lends itself more to field application.

The single sieve approaches for ballast-fouling intervention are needed because these methods provide the opportunity for more direct comparison of ballast condition to the surroundings. Understanding the variability of track conditions and the relationship of these variations to the local surroundings and site constraints can provide the basis for improved specification of field work operations. However, the goal for the industry is a continuous measure of ballast fouling along the track to provide a map of fouling condition. This can be provided by ground-penetrating radar, as described in Chapter 8. Typically, a combination of field mapping of ballast-fouling variations and field spot and laboratory testing to more definitively document fouling conditions will provide the best insight to the cause and source of ballast fouling as well as the required extents of remedial action.

Obtaining ballast samples from selected locations can provide useful information when interpreting ballast-fouling grain size variations. To assess the ballast general condition, it is often desirable to have samples obtained from both the loaded zone beneath the rail seat area of the ties and from between ties. Sample selection for assessment of in-track ballast must be done with care due to variability along the track that is dependent on ballast placement, density, and influences of subgrade and track structure variations, as well as traffic and maintenance. Recommended ballast sampling locations from Klassen et al. (1987) are shown in Figure 3.8. Subballast and subgrade samples should also be obtained, and these excavations can also provide useful information on the interface boundary location and profile.

3.1.5 Ballast layer behavior

Characterizing the aggregate material using the previous tests is often accompanied by assessment of the stress, strain, and strength behavior of ballast to provide insight into performance of the ballast as a layer. Assessment of ballast layer performance is a critical aspect of track structural design and performance evaluation.

3.1.5.1 Ballast layer strength parameters

Ballast deforms a small amount under each load cycle. Typically, this deformation is mainly elastic, but there is a small component of plastic deformation. Therefore, ballast layer performance is usually best defined in terms of a limiting deformation criterion. This is different from statically loaded geotechnical structures where strength is defined in terms of material failure and not deformation.
Ballast deformation can be due to settlement and particle rearrangement, ballast fracture/crushing, and ballast wear/fatigue. These different deformation modes combine by varying degrees to develop the overall ballast layer deformation. Further development of these criteria could be based on laboratory tests where the particle gradation changes are accurately measured and linked to repeated load performance of ballast under a variety of test scenarios. This would allow for the better definition of failure, better life-cycle cost prediction, and could lead to improved screening tests to aid in ballast selection. Modeling techniques to replicate and extend the use of the results from these tests are being developed using discrete element modeling and the finite element method.

When compared with an ultimate strength parameter that simply provides an allowable static load not to be exceeded, the utility of the deformation-based strength criteria to develop material properties and performance models becomes clear (Sussmann and Hyslip 2010). Deformation-based criteria represent in-service failure modes, whereas an ultimate strength parameter

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![Figure 3.8](image-url) (a–c) Ballast sampling location recommendations.

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is not directly related or proportional to common track failure mechanism under normal operating loads. Therefore, the links between ultimate strength criteria and screening tests for ballast quality are limited.

Although ballast performance is best defined by the repeated load behavior, the ultimate strength of ballast can be related to a friction angle based on the Mohr–Coulomb failure envelop friction angle, $\phi$, as described in Section 2.4.3. Clean ballast may have friction angles in the range of $40^\circ – 55^\circ$ (Table 3.6).

The value $K_0$ is the coefficient of lateral earth pressure, which is the ratio of lateral stress divided by vertical stress in the ballast. Under repeated loading of the ballast layer, the lateral stress tends to accumulate and remain even after the vertical load is removed. The magnitude of residual stress in ballast relates to the cohesion intercept on the $y$-axis in a Mohr–Coulomb stress diagram. For ballast, the cohesion intercept is often taken as zero because it is a noncohesive material. However, when properly densified to the normal compacted ballast state, the behavior can include a cohesive component due to large interparticle compressive forces induced from repeated loading. This residual stress (Selig and Waters 1994) that develops in the ballast can produce a $K_0$ as large as 10 or greater.

The material properties of degraded, fouled ballast are expected to reflect a reduced interparticle frictional resistance and lower layer resilient modulus than the values for clean ballast shown in Table 3.6. However, there is a lack of available information to indicate these degraded property values for fouled ballast.

One major point of distinction is whether the fouled ballast is wet or dry. Wet, fouled ballast tends to have reduced strength and increased deformation under load when compared with clean ballast. Conversely, dry, fouled ballast can have increased strength and stiffness; however, this may not translate into reduced, repeated load deformation or settlement when compared with clean ballast. This dry, fouled ballast behavior is caused by specifics of the sample related to particle size, gradation, initial density, plasticity, and cementitious characteristics of the fouling material.

After being disturbed by tamping, fouled wet ballast is expected to resettle faster than clean ballast. Although it is true that the settlement rate of ballast is influenced by the fouling material, it should not be assumed that the ballast particles lose contact with each other with increasing fouling, nor that the deformational characteristics of the fouled ballast become those of

<table>
<thead>
<tr>
<th>Property</th>
<th>Clean</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friction angle, $\phi$</td>
<td>$40^\circ – 55^\circ$</td>
</tr>
<tr>
<td>$K_0$</td>
<td>1–10</td>
</tr>
<tr>
<td>Resilient modulus, $M_R$</td>
<td>140–550 MPa (20–80 ksi)</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.3</td>
</tr>
</tbody>
</table>

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the fine material in the interparticle voids of the ballast. After tamping and recompaction of fouled ballast under traffic loading, the ballast particles will be forced back into contact, thus diminishing the effect of the fouling material on the mechanical behavior of the ballast structure. If the point contacts between ballast particles can be re-established by traffic compaction, the layer becomes increasingly resistant to deformation with further repeated loading. Under these conditions, the settlement trend of fouled ballast is expected to increase at a decreasing rate and become more stable with deformation rather than increase at an increasing rate as would, for example, a plastic clay that becomes progressively weaker with repeated load cycles.

Understanding the strength and variability of clean and fouled ballast is important for developing new repair strategies, improved maintenance procedures, and assessment of track load capacity and track life for maintenance planning.

### 3.1.5.2 Ballast stress–strain behavior

The bulk density or unit weight of ballast affects strength, with increased density leading to increased strength. Selig and Waters (1994), Lambe and Whitman (1969), and others clearly show that both the angle of internal friction and the deviator stress at peak \( (\sigma_{1f} - \sigma_{3f}) \) increase with increasing density (decreasing void ratio). As with any granular material, the strength increase is attributable to volume change behavior, that is, dilatancy, where denser material at the same confining pressure will have higher peak strength than a less dense material at that same confining pressure. The less dense material tends to force particles into a tighter packing when loaded, where a dense material cannot allow particles into a closer spacing without requiring the particles to dilate or pass over one another, resulting in an increase in volume to accommodate deformation. Much of this behavior is stress dependent and a result of the nonlinearity of the failure envelop.

Loose ballast tends to have a unit weight of 90–100 lb/ft\(^3\) (1.44–1.6 Mg/m\(^3\)), while that for compacted ballast is approximately 110 lb/ft\(^3\) (1.76 Mg/m\(^3\)) (Table 3.7). In Table 3.7, fouling index refers to the amount of ballast contamination. While this is a small change in unit weight, the associated engineering behavior changes distinctly. As reported in Selig and Waters (1994), Knutson

<table>
<thead>
<tr>
<th>Ballast condition</th>
<th>FI</th>
<th>Unit weight (lb/ft(^3))</th>
<th>Void ratio</th>
<th>Void volume (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose</td>
<td>0</td>
<td>95</td>
<td>0.68</td>
<td>41</td>
</tr>
<tr>
<td>Compact</td>
<td>0</td>
<td>110</td>
<td>0.53</td>
<td>35</td>
</tr>
<tr>
<td>Moderately fouled</td>
<td>20</td>
<td>125</td>
<td>0.35</td>
<td>26</td>
</tr>
<tr>
<td>Heavily fouled</td>
<td>40</td>
<td>135</td>
<td>0.25</td>
<td>20</td>
</tr>
</tbody>
</table>

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and Thompson (1978) noted large permanent deformation associated with loose ballast that had a unit weight of 90 lb/ft³ (1.44 Mg/m³) and almost negligible axial strain comparatively for ballast that had unit weight of 107 lb/ft³ (1.71 Mg/m³), confirming the trends of large differences in ballast behavior with compaction.

In order for ballast to densify, the stress state must be adequate to force the ballast into a contractive state. As shown in data reported by Indraratna and Salim (2005), the stress state has a controlling influence on the change in ballast behavior from dilative at low-confining pressure to contractive at high-confining pressure (Figure 3.9).

Compaction from repeated cycles of traffic loading is required in the field to densify ballast after maintenance to ensure track stability. The increase in stiffness and strength observed to develop after a large number of train passes is attributed to increased packing density of the aggregate mass. In Figure 3.10, the data from Suiker (2002) show the general trend of the effect of increasing packing density on ballast deformation response. Ballast densifies if the applied loads are adequately large to force the particles together. At low applied stress ratio \( n = \sigma_s / \sigma_3 \), the data curve labeled as “2” in Figure 3.10 shifts from the behavior of the virgin, as placed material, but does not change significantly. However, with higher loads, the behavior after 1 million load cycles changes

![Figure 3.9 Ballast behavior in triaxial test. (Idealized behavior based on data from Indraratna, B. and Salim, W., Mechanics of Ballasted Rail Tracks, A Geotechnical Perspective, Taylor & Francis, London, 2005; Selig, E.T. and Waters, J.M., Track Geotechnology and Substructure Management, Thomas Telford, London, 1994.)](image-url)
dramatically. The data curves labeled as “3” and “4” in Figure 3.10 show an increase in stiffness and peak strength. Suiker (2002) attributed this increase in peak strength to an increase in density.

The strong relationship between ballast unit weight and strength has been amply demonstrated (Hay 1982; Selig and Waters 1994, among others), but a practical ballast density measurement device or technique has not been developed. Part of the challenge in assessing ballast density arises from the large particle size and the required sample sizes to obtain a representative measurement. Non-nuclear density gages may provide the needed data to better quantify the influence of ballast density and associated strength changes on ballast performance and to develop associated quality control criteria.

Ballast grain size distribution has an effect on ballast deformation characteristics. The effect of grain size distribution on strength can be attributed, at least in part, to void ratio changes and changes in the particle interlocking behavior, which is known to affect the propensity for dilation/contraction. Roner (1985) found that differences in “parallel gradations” (i.e., which have a fixed gradation curve distribution shape but are scaled uniformly up or down in grain size) have essentially the same friction angle when compared at the same void ratio. This finding highlights the need to compact ballast (reduce the void ratio) to increase ballast layer strength.

### 3.1.5.3 Ballast deformation

In addition to strength parameters, ballast characterization must include a definition of the deformation behavior. Resilient deformation and resilient modulus are important parameters required for analysis of track performance and load distribution throughout the track structure. While Young’s
Modulus describes stiffness as the slope of the stress–strain diagram, ballast stiffness under repeated loading is described by the resilient modulus, $M_r$, where the slope of the stress–strain diagram is taken after the material has stiffened under an initial series of load cycles.

The deformatiional behavior observed in the static triaxial compression test can be characterized by the tangent or secant modulus, taken as either tangent to the stress–strain curve in the early portion of shear or as the slope of the line through zero. One problem with these definitions is that the strain experienced during shear is composed of both an elastic and plastic component, and there is no way of distinguishing between the two in monotonic loading. The second problem is that the modulus changes with the number of loads, with the most significant change after the first load cycle. This is attributed to densification and rearrangement of the ballast particles, sometimes referred to as shakedown. For these reasons, cyclic load tests are generally more desirable for evaluation of repeated load strain behavior and characterization of modulus at various stages of ballast life. After an appropriate number of load cycles, the differences between secant and tangent moduli often become insignificant. However, accurate assessment of ballast deformation and resulting track settlement can only be obtained based on knowledge of these changes derived from resilient modulus testing.

Other properties of ballast include Poisson’s ratio, which is rarely evaluated and often assumed to be 0.3, and $K_0$, which is also seldom measured but is often assumed to be 1, although under certain conditions it can be considerably greater for ballast as indicated in Table 3.6. These material properties coupled with an accurate assessment of ballast layer thickness are the common parameters used to describe ballast behavior in analytical models like GEOTRACK (described in Chapter 4).

### 3.1.5.4 Ballast layer residual stress

To provide the needed stress reduction to the layers below it, ballast must be stiff compared to lower layers. Figure 3.11 shows the ballast layer subdivided into three zones. In the upper zone, the load from the tie is applied to the ballast in the most typical mode of ballast loading: common triaxial compression. In the middle of the layer, the vertical load has reduced because of load distribution of the track structure, while the confining pressure increases. The lower layer shows the incremental tension that develops in the ballast under each successive wheel load. Failure is predicted in layered elastic models when the incremental tension exceeds the confining pressure. As the confining pressure in ballast is often low, very little incremental tensile stress can actually develop without failure, which often takes the form of particle spreading and rearrangement. Conceptually, the incremental tension predicted by the models in the bottom of the ballast layer results from the softer underlying layer tending to deform more than the
upper stiffer layer as described for pavements in Garber and Hoel (2015).

The deformed interface that develops at the bottom of the ballast layer is longer than the undeformed interface, indicating conceptually the tendency for ballast to spread near the subgrade interface.

The tendency for tensile stress to develop in the bottom of the ballast layer occurs during the initial process of ballast compaction or shakedown. When the incremental tension exceeds the confining pressure in this bottom ballast layer, particle spreading and rearrangement will occur in this zone. As the particles spread under tension, ballast particles from above fall into the spaces created. When the tension is released the particles that have fallen are wedged into position very tightly. Under repeated load, conceptually this becomes a ratcheting type of process where the incremental tension from each wheel load spreads the ballast, and particles above fill in the created space. When the load from the wheel is removed, the particle is held tightly in position. This occurs until the combination of the compressive stress that interlocks the particles and the confining stress exceeds the incremental tensile stress from loading. At this point, the incremental stress changes under load are still tensile, but the compressive stress in the layer, due to the interlocked stress between particles, is greater (Selig 1987). In this case, the total stress is still compressive even though the incremental stress may be tensile, thereby reducing the magnitude of the compressive stress. This high-interlocked stress (Uzan 1985) is most likely the mechanism by which large $K_0$ conditions develop, as has been observed in ballast tests (Selig and Waters 1994).

![Figure 3.11 Ballast layer loading resulting in the tendency for cyclic tensile (T) strain in a material with no tensile capacity.](image)
The interlocked stresses (Barksdale and Alba 1993) that develop have sometimes been termed residual stresses. The development of residual stress is only possible if the ballast is supported by an elastic layer that can deform under load and then return to its original position. If either plastic deformation occurs or the supporting layer does not return to its original position, residual stress cannot develop in the ballast, although ballast spreading may continue. Ballast residual stress is the main mechanism by which the ballast interlocks and compacts and is sometimes incorrectly referred to as consolidation in lateral stability-related research documents.

The residual stress in the layer that develops under this process of rearrangement is unique and represents a significant stress level. In bound layers such as cement or asphalt where particles cannot rearrange to accommodate this incremental tensile stress, the strength of the material must be greater than the incremental tensile stress. In a similar manner, use of reinforced layers like geogrid or geoweb must also be designed with an understanding of this behavior. Reinforced layers can perform adequately, but compaction will be critical so that the particles will not be able to rearrange freely to build up residual stress because this process might damage the reinforcing material.

3.1.5.5 Ballast strength and deformation tests

Assessment of ballast performance is an important aspect of track structural design and performance evaluation. However, testing of ballast performance is often challenging due to difficulties in obtaining representative samples and preparing and performing tests that accurately represent field conditions.

3.1.5.5.1 Direct shear test

The shear strength of ballast, obtained from a strength test such as the direct shear test (Figure 3.12) or the triaxial compression test, is typically

Figure 3.12 Large direct shear apparatus. (Courtesy of University of Illinois Urbana-Champaign, Champaign, Illinois.)
characterized by the stresses at failure and reported as the friction angle, $\phi$, as described in Section 2.4.3. A shear strength assessment provides a good indication of the ballast friction angle and, as such, may be a suitable screening test, but ballast rarely fails with a well-defined shear failure plane, and so this test does not represent a failure mechanism that is common for ballast.

3.1.5.5.2 Triaxial test

The triaxial test is often more suitable for ballast characterization. The evaluation of ballast strength with a triaxial test can provide both the friction angle and Young’s modulus (stiffness). Repeated load (cyclic) triaxial tests provide the resilient modulus and the friction angle. The resilient modulus is an important parameter in ballast assessment because the ballast often stiffens with increasing load cycles (Figure 2.18). The stiffness attained by the ballast after it stabilizes following a large number of repeated load cycles is a more meaningful parameter than is the initial stiffness.

Because ballast is generally well-drained and typically does not develop excess pore pressure during compressive testing, it is often assumed that the total stress is equal to the effective stress. Although it is possible that excess pore water pressure may develop as ballast becomes increasingly fouled, the amount of fouling corresponding to this condition is likely to be impractical. Tests of fouled, wet ballast should identify whether effective stress or total stress analyses was being used.

Triaxial testing of ballast requires a large test apparatus as shown in Figure 3.13. Typically, the diameter of the aggregate sample being tested should be at least 2.5–3 times greater than the largest individual particle size according to ASTM standards. The triaxial test can be used in monotonic loading to determine the ultimate shear strength of ballast. However, as the load on ballast does not monotonically increase until failure, a better use of the triaxial test is for repetitive load testing, where the samples are tested for particle breakage and permanent deformation after a large number of load cycles. This test method most closely resembles the load environment and failure mechanisms of ballast in track. Use of the repeated load triaxial test is recommended for ballast evaluations that include lifecycle cost–related ballast performance parameters of strength, stiffness, and deformation.

3.1.5.5.3 Ballast box test

Over the years, various boxes have been created for lab tests to simulate the tie–ballast interface. Investigations have been conducted into the influence of the type of box, material, and associated measurements to ensure that the influence of significant parameters can be measured within the
limitations of the device (Bennett et al. 2011). The ballast box design must consider the stiffness and frictional resistance of the sides to ensure reasonable fidelity with field performance observations. A bigger box is generally desirable to limit the influence of the edges of the box, but there is also the concern that bigger boxes require more sample preparation, setup, and applied load. The ballast box provides a way to overcome the limitations of more traditional lab tests such as the triaxial cell, which does not consider the tie–ballast interaction. This interaction is best assessed with a physical simulation like a box test (Figure 3.14) coupled with analysis and modeling.

The influence of fouling on the mechanical behavior of the ballast can be estimated from a triaxial test. However, this test cannot assess the influence of hanging ties on track settlement and how this affects required maintenance. Ballast settlement routinely causes track settlement and track geometry anomalies, but sometimes it results in the ballast settling away from the bottom of certain ties leaving these ties hanging. Selig and Waters (1994) present data to indicate that ballast settlement increases slightly with increasing fouling index. However, when a gap is introduced between the tie and fouled ballast to model a hanging tie, the ballast settlement increases by a factor of approximately 4 or more at the same fouling index (Figure 3.15).

The ballast box tests provide evidence that hanging ties damage track and cause more settlement than well-supported ties, but the real benefit of
this test is that it provides the ability to quantify the damage. This quantification can support planning and justification for maintenance to limit damage from hanging ties, and it points to the value of instrumenting the tie section for load to evaluate if the tie is structurally suitable for the applied loading. Although this type of test opens up many possibilities, care must be exercised because these are not standard tests and success is dependent on understanding the problem and experience with this type of testing.

Further testing by Han (1997) developed detailed relationships between various ballast parameters and expected performance. This analytical tool was based on significant testing of ballast to develop data on ballast settlement for many conditions, an example of which is shown in Figure 3.16.
3.1.6 Ballast compaction

Compaction of ballast, sometimes colloquially referred to as consolidation, typically results from traffic loading but can also be provided in a more controlled way by application of the dynamic track stabilizer to help ensure lateral track stability after maintenance that disturbs the ballast. As compaction changes the unit weight or density of the material, it is common practice to use a measure of unit weight to judge the level of compaction of most geomaterials. However, measurement of ballast unit weight is challenging due to the large-sized aggregate (Yoo et al. 1978). Nuclear density probes are sometimes used, although the disturbance produced by the probes can influence the measurement. In-place tests of unit weight using the water replacement technique (Panuccio et al. 1978) is probably the most reliable method, although this is very time consuming and results in a small number of tests. Ballast settlement potential can be very sensitive to even small changes in its density due to compaction, degrading ballast condition, or track maintenance. The small range of expected results coupled with the dramatic changes in performance demonstrates the need for accurate data to predict ballast settlement.

Density testing was developed as a measure of the soil physical state indicative of its mechanical properties. The measured density of soil is often a simplified way to estimate its mechanical strength or stiffness because density is usually easier to measure than tests that measure strength or stiffness directly. As ballast density tests can be more difficult in ballast than field mechanical strength or stiffness tests, several mechanical tests have shown promise for field ballast assessment including plate load tests used on the Channel Tunnel Rail Link (O’Riordan and Phear 2001). On this

![Diagram](image-url)
project, the plate load test data were correlated with unit weight to provide the more common density terminology for specification and discussion, while using the stiffness criteria for field assessment. Further discussion of the plate load test is provided in Chapters 8 and 10.

The process of ballast compaction involves an interlocking of the particles into an increasingly tighter arrangement with increasing number of repeated load cycles. When this process works properly, the compacted ballast layer becomes strong enough to support the applied loads without significant plastic deformation while providing resiliency so that the ballast absorbs some of the applied energy. The compaction process is necessary to provide interlocking of ballast particles and increased ballast layer stiffness to substantially reduce the stress transmitted to the lower layers.

The interlocking process relies in part on a ratcheting mechanism that involves deformation of the ballast layer under load with the tendency of the applied load to spread the bottom of the ballast layer. This tendency for temporary spreading reduces the ballast interparticle contact force allowing the particles above to slip into a tighter packing with the particles below (Selig et al. 1986; Selig and Waters 1994). As the load is removed, the tendency for spreading is removed and the interparticle contact force returns at an even larger magnitude, provided the ballast below is confined and not allowed to deform plastically. Without adequate confinement, the bottom ballast will tend to spread and this interlocking process cannot occur. With too much confinement, the interparticle force will be too high and cannot be reduced enough to allow the particles to slide past one another into a tighter packing. This is one of the basic mechanisms likely resulting in the optimization of track stiffness for track performance longevity (Hunt 1999). After a period of time, the ballast compaction process reaches equilibrium as the tendency for spreading of the ballast will not reduce the interparticle contact force enough to allow further interlocking. As long as the loading stays within the range of applied loads during the compaction process, the ballast will be stable. However, any significantly higher loads would lead to further compaction.

It is important that the compacted condition of the ballast be restored following maintenance disturbance to minimize settlement to provide track lateral strength and buckling resistance. Compaction under traffic loading can provide this, but a more controlled and quicker compaction can be provided using a track stabilizer after tamping. The track stabilizer is designed specifically to provide quick on-track ballast compaction (Figures 1.33 and 3.17). The amount of compaction provided by the stabilizer is not generally known and is difficult to estimate. The stabilizer has a head (Figure 3.17) that grips the track laterally, applies a downward force, and provides a 25–42 Hz horizontal vibration to rearrange and compact the ballast. The amplitude and frequency of the vibration coupled with the magnitude of the downward force determine the amount of compaction that can be achieved, and this varies somewhat along the track due to variable support...
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and track condition. Although a track stabilizer can only provide a small percentage of the needed ballast compaction, the compaction it does provide is generally more uniform and controlled compared to ballast compaction from traffic (Sussmann et al. 2003b).

As stated, the ballast compaction achieved using the dynamic track stabilizer is expected to vary depending on the track and field conditions. Lateral track strength test results on concrete tie track have shown increased lateral resistance following track stabilization (Sussmann et al. 2003b). However, the stabilizer might not be optimized for the particular track and substructure characteristics under test, and the performance of the stabilizer should be verified with field test data to ensure good results.

Obviously, providing a controlled density of ballast in track is beneficial. Techniques are being developed to provide monitoring of the compaction process (intelligentcompaction.com) for highways and pavements, with a minimum of field testing needed for verification. Intelligent compaction provides a direct measurement of the soil stiffness based on the response of the vibratory compactor. This effort recognizes the benefits of uniformity of support and of properties. Typical compactors for aggregate and subgrade compaction use a single drum roller (Chang et al. 2011), which can be modified to monitor compactor response to assess the compaction of aggregate. For newly constructed track, the criteria for proper placement of ballast has been assumed to be ballast density where a minimum density could be specified. By correlating the stiffness of support and its effect on the vibration frequency of vibratory rollers, intelligent compaction

Figure 3.17 Dynamic track stabilizer head. (Courtesy of Rainer Wenty, Plasser & Thuerer, Vienna, Austria.)
technology could provide an improved density specification and control for track. Historically, the challenge has been to develop a ballast density measurement. However, new concepts in measurement of the compactor response coupled with improved measurements of the stiffness of placed materials and track deflection testing may provide the opportunity to measure track response to load as each layer is constructed. In this way, the construction specifications can be written in terms of track deflection and could be verified by a deflection test of each layer prior to acceptance.

Ballast compactors have been tried over the years with reasonably good compaction results, but their use in those early trials lacked economic justification due to an overabundance of track capacity that limited the benefit of reduced slow orders on train operations.

The subject of compaction, in general, is not complete without at least a brief discussion of proof rolling. To ensure proper compaction in applications such as high tonnage and high-speed lines, as well as such critical locations as bridge transitions and grade crossings, it is recommended that the design be verified by proof rolling. Proof rolling is common practice on many building project sites and a similar application on track to compact the old exposed roadbed on which the new track will be constructed to reveal weak spots that require over excavation and replacement, reinforcement or drainage. If these weak spots are adequately addressed, this will result in a more uniform platform for track construction. It is recommended that the proof rolling concept be employed to ensure adequate track support from lower layers in construction and maintenance operations that remove the track structure and expose the lower layers.

### 3.1.7 Used ballast

Used ballast typically refers to the reclaimed material returned to track during the undercutting cleaning operation. As the finer material is removed, only the larger particles are returned to track during undercutting/cleaning. Used ballast is typically smaller in size than the specification for new ballast, and the wear and abrasion under traffic that reduced the particle size usually reduces the particle angularity, especially when water is present. Although newly broken ballast from track would be angular, continued wear under traffic is expected to develop worn ballast that is not as angular as new ballast, and this reduced particle angularity may affect its deformational behavior somewhat under loading.

Generally, the amount of old ballast with particle sizes large enough to be returned to track constitutes less than half of the ballast. Therefore, the majority of the ballast will be new. If the used ballast could be mixed with new ballast, the distribution of a small proportion of less angular used ballast particles into a new ballast layer would not be expected to be particularly detrimental to performance. Moaveni et al. (2013) describe application of a method to assess ballast in the field so that properly graded
and shaped particles are placed to ensure ballast performance. However, ballast-handling procedures during undercutting do not allow for this type of used and new ballast mixing during placement. Rather, the used ballast is placed in track as a distinct layer below the new ballast and above the subballast with new ballast placed on top.

As a distinct layer, Indraratna and Salim (2005) found that the peak principal stress ratio was reduced about 60% from the level measured for new ballast at low-confining pressure of 10 kPa. The difference between new and used ballast was found to be negligible at a confining pressure of 200 kPa (Indraratna and Salim 2005). Therefore, with adequate confinement a used ballast layer can perform well in track. Means to confine the used ballast might be provided mechanically by soil reinforcement, but a substantial amount of confinement will be provided simply by placing the used ballast as deep in the track section as possible and confining the used ballast to the center of the track to avoid the low-confinement conditions in the shoulder region.

### 3.2 SUBBALLAST

Subballast is the granular layer that is located below the ballast and above the subgrade, which has been either placed as a specific layer or evolved in-place from the particle wear, densification, and settlement of old ballast layers due to decades of loading and track maintenance. The latter condition is very typical of railway lines that have been active for a long time and where the old roadbed acts essentially as a subballast layer.

As a structural layer, subballast reduces stress to the subgrade, similar to ballast, by an amount dependent on its resilient modulus (stiffness) and thickness. Subballast stiffness generally controls the load-spreading ability of the subballast layer and depends largely on its compacted density, which in turn is controlled by its gradation. The typically well-graded subballast allows a high relative density and stiffness, but the narrowly graded ballast is stiffer due to the dominating effect of interlock of the larger particles.

Subballast must not plastically deform over many load cycles. This requires subballast to (1) be well drained and avoid positive pore water pressure under repeated load and (2) have durable, angular particles that interlock and resist abrasion.

Under well-drained conditions, the strength and settlement characteristics of the subballast will be governed primarily by the density of the material, provided that durable and angular gravel is used. Although broadly graded subballast has the advantage of achieving a high-compacted density, strength, and layer stiffness, an overabundance of fine particles will inhibit drainage and potentially lead to excessive settlement under loading if saturated (see Chapter 6). For the subballast layer to be stable under loading, its gradation should be designed to be well drained as the first priority, and
secondarily optimized for density and ease of compaction after considering strength and stiffness requirements.

The strength and settlement characteristics of subballast are affected by the amount of moisture in the aggregate. In an unsaturated condition, the presence of a limited amount of water acts to “lubricate” the particles and increase both elastic and permanent deformation, even without developing pore pressure (Thom and Brown 1987). High fines content is undesirable because fines tend to both attract and retain moisture, which causes problems due to low permeability and the possibility of retaining water within the subballast layer and in contact with the subgrade.

3.2.1 Subballast functions

The subballast is a select crushed stone or gravel and sand mixture that is used to cover the natural subgrade soil or fill material (on an embankment) to provide a solid and well-draining layer under the track. As an intermediate gradation generally between the fine subgrade and coarse ballast, subballast must be designed to separate these layers to prevent intermixing.

The primary functions of subballast are to (1) provide drainage out of the track, (2) help reduce applied stress to the subgrade, (3) provide separation between the ballast and subgrade, and (4) help to provide frost protection to the subgrade in colder climates.

3.2.2 Subballast characterization

The most important parameter for characterization and selection of subballast is the grain size distribution of the material because of the influence of gradation on the separation and drainage functions. The most significant characteristic of the subballast gradation is the fines content, which is the amount of material in the silt to clay size of <0.075 mm. When subballast has a fines content of approximately 10% or more and is in the presence of water, it becomes susceptible to excessive deformation under cyclic loading, and it tends to hold/retain water through capillary tension. Subballast with low fines content will typically have good drainage characteristics, but might not provide the needed separation between subgrade and ballast.

Small increases in fines content can dramatically change the drainage characteristics of subballast. The permeability and water retention characteristics of granular aggregate are strongly influenced by the size of the individual void spaces. Smaller average void space results in an increased tendency to retain water in the material due to lower permeability. A US Federal Highway Administration study (Tandon et al. 1996) of the water retention capacity of subbase found that higher fines content leads to higher subgrade saturation levels. It was noted that small amounts of fine material (passing the #200 sieve, 0.075 mm) in the road base material drastically
affected the amount of water retained. In two comparable densely graded base samples, the water retention doubled from 41% to 82% saturation with an increase in fines from 1.6% to 5%.

The AREMA manual specification for subballast limits the percent passing the #200 sieve to 5%. However, subballast with 5% fines may still remain nearly saturated (Tandon et al. 1996). To provide free-draining subballast, the data from Tandon et al. suggest that 2%–3% or less of fines will be required.

The drainage requirements must be weighed against the filtration/separation criteria presented in Section 6.4.2. The conflicting drainage and separation requirements must be balanced. This becomes especially critical on fine-grained subgrade where drainage is equally important to separation criteria. The fine sand portion of the subballast is needed to provide the required separation of the subgrade from penetrating the ballast, but the fines portion should be as low as possible to provide proper drainage characteristics.

A subballast gradation that conforms to both the drainage and separation functions will generally have good strength characteristics, provided that the material has been compacted properly and has adequate angularity and interparticle friction. The typical range of subballast properties for friction and other parameters are shown in Table 3.8.

Due to exposure to a harsh environment, the subballast must be comprised of material with high particle durability and low environmental reactivity. The durability and reactivity are functions of material type and are measured as described in the section on ballast. The AREMA manual recommends performing LAA (ASTM C131) and sodium sulfate soundness (ASTM C88) tests on subballast but does not specify values for acceptance. Reasonable limiting values for subballast acceptance of LAA and sodium sulfate soundness tests are <50% and <5%, respectively. Subballast material must not react with conditions in the environment that produce cementing, expansion, or deterioration. ASTM D2940 and D124 help to ensure that subbase material for highways will not break up when subjected to freeze-thaw or wetting-drying cycles and that the subbase has a LAA value no greater than 50. These are also desirable characteristics for railroad subballast.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friction angle, $\phi$</td>
<td>25°–40°</td>
</tr>
<tr>
<td>$K_0$</td>
<td>0.4–1</td>
</tr>
<tr>
<td>Resilient modulus, $M_r$</td>
<td>55–105 MPa (8–15) ksi</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>0.3–0.4</td>
</tr>
<tr>
<td>Permeability (minimum)</td>
<td>100 m/day</td>
</tr>
</tbody>
</table>
3.2.3 Subballast performance

The effect of moisture on performance of granular material will be described in subsequent sections as a softening of the material, one that provides a mechanism for increased deformation. Both elastic and plastic deformation are influenced, but moisture increases the plastic deformation more dramatically.

In service, when subballast becomes saturated and subjected to cyclic loading, increased pore pressure can develop that can lead to severe loss of strength and high amounts of deformation, through mechanisms of liquefaction or cyclic mobility. This type of saturated failure of subballast was documented at the Association of American Railroads (AAR) test facility in Pueblo, Colorado (see Read and Li 1995 and Chapter 6). Subballast should have low fines content and reasonable thickness not exceeding approximately 12 in. (300 mm) to allow for quick drainage and to prevent pore pressure development and subsequent settlement. However, as stated, the separation function requires the subballast to have a certain amount of fines to protect ballast from penetration of finer soil below.

The gradation and unit weight (in-place density) of subballast have the biggest influence on its performance. An investigation by Thom and Brown (1987), which included a range of gradations from uniform gravel down to a well-graded soil containing nearly equal parts of silt, sand, and gravel showed that the compacted unit weight had more of an effect on strength than did gradation. Gradation was found to have more of an effect than density on permeability. Therefore, selection of subballast gradation is critical to track drainage considerations, and so gradation should be considered as comparable in importance to the density–strength relationships required for structural performance of the subballast layer to ensure proper long-term track performance.

3.2.4 Subballast drainage

A clean and free-draining ballast layer underlain by well-draining subballast is necessary for good substructure stability and performance. If the ballast layer is properly maintained to be free-draining, then water that enters the track from above by precipitation is drained out of the track by the ballast and subballast layers (Tandon et al. 1996). Subballast functions to provide drainage of water from downward infiltration from the ballast, lateral infiltration from side of track, and upward migration from the subgrade. In most cases, the predominant source of water to the track substructure is from downward infiltrating water from the ballast.

Granular subballast layers do not function well as water-shedding layers until they saturate. Heyns (2000) found that even for relatively impermeable subballast (permeability of 0.2 to 0.6 m/day) the subballast layer does not shed water across the subballast/ballast interface, unless the rainfall rate is sufficiently high. This water-shedding off the top of the subballast layer only occurs when the rainfall intensity is in the order of 25–50 mm/h for typical
subballast gradations. For low-permeability subballast (e.g., subballast with 20% fines), only in extreme rainfall events (>70 mm/h) does water shed off the top of the ballast/subballast interface. The testing also concluded that the slope of subballast/subgrade interface does not play a significant role in avoiding subballast saturation. Rather, the interface slope is a significant factor for estimating the time for drainage of the subballast once the rainfall stops.

As found by both Tandon et al. (1996) and Heyns (2001), it is more common for subballast with fines to saturate during rainfall with the fines holding the moisture longer after the rainfall event, which would lead to more subgrade saturation. To avoid subgrade saturation, an impervious seal of the subballast material surface would provide the best method to shed water over the surface. Alternatively, well-draining subballast with very little fines could be used to minimize the time required to drain the subballast layer.

As water tends to infiltrate the subballast layer rather than being shed laterally at its surface, it is necessary to ensure that the subballast material is as permeable as possible considering the constraints imposed on the grain size distribution by the separation criteria. At a minimum, this would require establishing and maintaining a free-draining end of the subballast layer immediately at the toe of the ballast shoulder or establishing a zone of free-draining material outside the edge of tie. Further details on track drainage design considerations are included in Chapter 6.

### 3.2.5 Separation

Subballast provides an essential function of separating the ballast and subgrade. The grain size distribution for subballast must prevent intermixing of ballast and subballast, and the migration of subgrade into the ballast. As depicted in Figure 3.18, the soil particle at location “A” is carried with water into the subballast above. If the subballast grain size distribution is properly sized for the subgrade, the soil particles will become trapped in the lower portion of the subballast and will not travel into the material. The filtration and separation criteria are described in Chapter 6.

![Figure 3.18 Soil migration and separation/filtration.](image-url)
Although subgrade fouling of ballast is not the most common source of fouling because an adequate separating subballast layer is usually present, one of the cases where this does occur is through subgrade attrition where ballast is placed directly over a hard but erodible subgrade material such as a hard clay or soft rock. Attrition is the wearing away of a hard subgrade surface by ballast grinding on it in the presence of water that produces muddy fines that are subsequently squeezed up into the ballast by cyclic train loading. Subballast, if present, acts to cushion the hard ballast particles from the erodible subgrade surface and thus preclude subgrade attrition.

For conditions of water seepage through soil, the soil gradation requirements for separation, clogging, and filtration can be determined based on the separation criteria outlined by Terzaghi, and more fully developed by the US Army Corps of Engineers and the US Bureau of Reclamation. Cedergren (1989) provides thorough coverage of this topic. The specific guidelines and their application, including application to geosynthetics, are further covered in Chapter 6.

### 3.3 SUBGRADE

Subgrade, the foundation on which everything above depends for support, is often the most variable and potentially the weakest of track components. The inaccessibility of subgrade makes it challenging to assess its condition, diagnose a problem, and prescribe or implement a remedy with confidence. Changes in soil type, properties, and physical state can all occur over short distances of tens of meters making identification of problematic conditions a particular challenge due to the long distances involved. Understanding subgrade behavior provides the basis for the investigative tools and remedial methods that have been developed.

The term *subgrade* is sometimes used to denote the elevation grade line of the track ready to receive subballast during construction. This tends to oversimplify the nature of subgrade and minimize the distinction of the various subgrade elements that often are defined as the formation and fill. In a cut, the subgrade is usually exposed natural soil or rock, which is often considered the formation layer. The formation layer is most commonly defined as the natural soil layer supporting the track. In a fill, the subgrade surface is usually the top of the embankment constructed of locally available fill on top of the formation layer. At grade, subgrade is usually natural soil and a thin layer of placed fill to smooth out local variations in elevation. If the subgrade is granular, then track stability is usually assured and drainage becomes the main concern, although the subgrade will not be overly sensitive to moisture variations. If the subgrade is fine grained, then questions of stability can arise requiring some classification of the subgrade and drainage becomes even more of a concern, especially for moisture-sensitive clays.

Unlike buildings or other structures with a limited footprint, track right-of-way, like all transportation infrastructure, is a low-cost per foot structure.
that has not historically justified a detailed site investigation. The maintainable nature of track has been used to argue against the need for a site investigation, and for the approach of relying on train traffic to eventually find the weak subgrade zones for maintenance forces to repair, using a “build now, fix later” approach.

However, this “fix as needed” approach to subgrade has its limitations. Some desired subgrade features may be very expensive to restore once they have deteriorated because of the costs of reduced track availability during maintenance and cost of the work. For example, the drainage cross-slope is needed to avoid trapped water on the subgrade and to rapidly drain the subballast, but re-establishing the cross-slope once subgrade has settled and deformed requires removal of the track structure and taking the track out of service for a period of time. In most mainline situations, track time for maintenance work windows and train delay time due to slow orders are expensive opportunity cost items. The subgrade repair and maintenance costs should be considered in light of present-day opportunity costs of maintenance, repair, and rehabilitation work items and train delay scenarios to develop an economically justifiable site investigation budget. Remote sensing technology currently provides a means to cost-effectively identify zones of subgrade variations that can be targeted with limited site investigation and reconnaissance to characterize the subgrade, develop appropriate track designs, and reduce the likelihood that subgrade deformation will affect track performance.

3.3.1 Subgrade functions

To function as a stable foundation layer, the subgrade must be structurally sound and not sensitive to environmental damage. Structural considerations for the subgrade include ensuring the subgrade is:

1. Stable under self-weight such that it does not deteriorate through consolidation settlement or massive track instability, sometimes referred to as massive shear
2. Stable under train loading and does not strain plastically to form a progressive shear failure known as a subgrade squeeze and does not deform and produce ballast pockets

The track must be designed to also protect the subgrade against environmental problems to minimize the effects of frost heave and shrink–swell-related volume change. Finally, the track must be designed to protect hard but erodible subgrade from attrition loss due to sharp-edged ballast particles grinding on it in the presence of water Selig and Waters (1994).

3.3.2 Soil types

The subgrade generally consists of any naturally occurring soil including the coarse soil types of sand and gravel and the fine-grained soil types of
silt and clay. Construction over previously developed areas may lead to construction debris and other by-products of development in the subgrade. Natural soils include both organic and inorganic soil masses, with the general recommendation that organic soils be removed and replaced. Although track has been built over existing organic subgrade deposits like peat, this is not a common or desirable subgrade material.

Subgrade soil characterization should include determination of soil type, grain size distribution, physical state, and mechanistic properties of strength and stiffness. The primary governing issue for subgrade is the soil type. This is followed by determination of the physical state of the soil including the density and moisture content. Finally, specific questions about the physical properties and engineering characteristics should be investigated.

Coarse-grained subgrade (sands and gravels) with little (<5%) clay or silt tends to be least problematic. With adequate drainage and reasonable surface compaction, most sand and gravel subgrade tends to perform well. However, coarse-grained subgrade is not trouble free and is subject to unique failure mechanisms including liquefaction and cyclic mobility. The grain size distribution, unit weight, water table elevation, and moisture content tend to be the main indicators of potential problems with liquefaction or cyclic mobility, although specific evaluation of engineering properties may be needed for design or remediation.

Clays and silts tend to be the focus of most subgrade improvement work because these types of soils are subject to the common subgrade failure modes of ballast pockets and subgrade squeeze. But clay- and silt-sized particles do not need to be the predominant constituent in a soil for these failures to occur. Even mostly coarse-grained subgrade with as little as 10% of clay or silt can take on the behavior of a fine-grained soil and be subject to these failure modes. Distinct subgrade behavior variations occur between silt and clay, making soil classification the first question when dealing with fine-grained subgrade. Once the soil type is identified, the physical state needs to be characterized because moisture content and degree of saturation, density, plasticity, and soil fabric are all important elements that might identify specific problems or highlight unique solutions. The depth of the water table and engineering characterization then become the next questions needed to identify problems or develop solutions or designs.

Organic soils such as peats are often the most problematic subgrade soils. Organic soils are identified by traditional soil characterization tests such as moisture content (possibly exceeding 100%) and grain size distribution. Specific testing for organic content and the state of decay of the organic material are also common. Peat is routinely recommended to be removed from the track subgrade, although this is not always practical for deep deposits or existing track built on organic soils. Methods to mitigate the impact of organic soils and peat include densification to strengthen the deposit and limit settlement, reinforcement of layers to reduce the applied stress to an acceptable level, or confinement to reduce the mobility of the peat.
Substructure 111

An indicator of subgrade performance is the amount of pressure it can withstand without deforming substantially. Although the AREMA design manual advocates a uniform minimum pressure for all North American subgrade, the properties of subgrade can vary dramatically and can provide widely varying levels of track support. Instead of considering the track as a structure that should provide equal support to the train for varying levels of subgrade support, the implicit assumption of the AREMA design procedure is that if the track is built to standards that protect against most problematic subgrade conditions, then the remaining problem zones can be repaired later as their locations become apparent by deforming under traffic loading (“build now, fix later”). However, once the track is in place and pressure to continue operations limits the track outage time, the options for repair of subgrade problems are often limited and costly.

The bearing pressure of subgrade soil is one parameter to consider when evaluating the variability of subgrade support conditions and when identifying subgrade that should be corrected prior to track construction. Existing problematic conditions are seldom known prior to construction without a detailed inspection of the track structure or local geologic conditions. Although conducting these tests during the planning stages for new track adds to the project cost, they provide more detailed understanding of conditions that can lead to a design with lower life cycle track cost by minimizing the expenditures over time.

3.3.3 Common problems

Unstable subgrade tends to mainly consist of fine-grained soil that often corresponds to lower strength and low permeability relative to coarse-grained subgrade. In general, more finely grained soil and/or greater plasticity soil tend to be found in poor performance zones. Figure 3.19 illustrates the

![Figure 3.19 Track settlement zone due to weak subgrade.](image)
signs of a poorly performing location with a fine-grained embankment that has settled under traffic.

Moisture content of the soil has a profound influence on subgrade performance. Most nonorganic soil types, even fine-grained soil, could function as stable subgrade if maintained at low moisture content. However, it can be very difficult in practice to limit the access that water has to moisture-sensitive soils. Sources of subgrade moisture include surface water infiltrating through the ballast and subballast, water in drainage ditches to the side of track that may infiltrate, and groundwater that may migrate up into the track. Yoder and Witczak (1975) indicate that groundwater can be a major subgrade concern if the water table is within approximately 6.1 m (20 ft) of the ground surface.

Stresses from loading can cause subgrade deformation under either the static weight of overburden or the repeated train dynamic loading, and one of these sources of loading will typically control the amount of deformation. The static weight of overburden material on a soft, deformable subgrade (Sluz et al. 2003) can cause consolidation settlement and, possibly, massive shear failure. Repeated traffic loading can be characterized by the magnitude of wheel loads, and their corresponding number of load cycles. A simple case is a bi-modal wheel load distribution consisting of unit trains of empty freight cars and unit trains of loaded freight cars. However, the actual wheel load distribution is typically more complex and should consider the range of static wheel loads including their dynamic augment as described in Chapter 2. The subgrade behavior under static load can be much different than under repeated loading because

1. Fine-grained soils will exhibit lower strength under repeated loading than under a sustained load of the same magnitude.
2. Small, plastic deformations under individual wheel loads can accumulate to form substantial track settlement.

It is necessary to characterize subgrade stability under both static and repeated loading cases.

Although far less common, unstable conditions can arise from coarse-grained soil that is saturated. In such a case, increased settlement of coarse-grained soils due to cyclic mobility or liquefaction can result from the increase in pore water pressure under repeated loading. In some anecdotal cases, this has been observed in the granular subballast layer as well.

Soil temperature is a concern due to cyclic freeze and thaw damage. Under certain combinations of temperature, soil suction, soil permeability, and availability of water, an ice lens will form when the water freezes, causing ground heave. When the soil thaws again excess water from the ice lens will be trapped in the subgrade and weaken the soil.

Table 3.9 summarizes subgrade problems in terms of loading, environmental conditions, and their causes and features. Several of these problems are discussed in more detail next.
The loading that drives massive shear failure is the substructure self-weight, and the weights from the train and track superstructure. The resistance to massive shear is from the shear resistance of the substructure layers. This type of failure often occurs shortly after track construction or

| Table 3.9 Characteristics of track subgrade problems |
|-----------------|-----------------|-----------------|-----------------|
| **Cause**       | **Type**        | **Factors**     | **Features**    |
| Dead load       | Massive shear failure | • Weight of train, track, and subgrade  
• Inadequate soil strength | • High embankment and cut slope  
• Often triggered by increase in water content |
| Dead load       | Consolidation settlement | • Embankment weight  
• Saturated fine-grained soils | • Increased static soil stress compared to before construction |
| Live load       | Subgrade attrition | • Repeated loading of hard subgrade by ballast  
• Contact between ballast and subgrade  
• Clay-rich rocks or soils  
• Water | • Muddy ballast  
• Inadequate subballast |
| Live load       | Cyclic mobility/liquefaction | • Repeated loading  
• Saturated silt and fine sand | • Large displacement  
• More severe with vibration  
• Can happen in subballast |
| Live load       | Progressive shear failure | • Repeated overstressing  
• Fine-grained soils  
• High water content | • Squeezing near subgrade surface  
• Heaves in crib and/or shoulder  
• Depression under ties |
| Live load       | Cumulative plastic deformation | • Repeated loading  
• Soft or loose soils | • Differential subgrade settlement  
• Ballast pockets |
| Environment     | Frost action (heave and softening) | • Periodic freezing temperature  
• Free water  
• Frost-susceptible soils | • Occur in winter/spring period  
• Rough track surface |
| Environment     | Swelling/shrinkage | • Highly plastic soils  
• Changing moisture content | • Rough track surface |
| Environment     | Slope erosion | • Running surface and subsurface water  
• Wind | • Soil washed or blown away |
| Environment     | Soil collapse | • Water inundation of loose soil deposits | • Ground settlement |
after heavy rainfall or flooding and is characterized by an abrupt loss of track alignment and surface.

When the subgrade is constructed as part of an embankment, the stress on the embankment foundation will increase due to the added weight from the embankment. If the embankment soil is not well drained, excess pore pressure will develop reducing the effective stress that can compromise stability, potentially causing a massive shear failure. The excess pore pressure dissipation and resulting settlement are analyzed in basic consolidation theory. In coarse-grained soils, the dissipation of excess pore pressure is rapid, and so most settlement is rapid and will occur during the construction of the embankment and track. In fine-grained soils, the dissipation of pore pressure is slow, resulting in settlement after construction.

Liquefaction and cyclic mobility are dynamic loading–related soil deformation mechanisms. These types of failures are the result of a loss of shear strength under repeated loading due to increased pore water pressure. Liquefaction occurs when loose, saturated cohesionless soils (particularly coarse silts and fine to medium sands) are dynamically loaded or vibrated. Under vibrated loading, soil particles tend to compact, transferring intergranular stress into pore water pressure. Liquefaction occurs when the pore water pressure exceeds the total stress in the material, resulting in zero effective stress and complete loss of frictional shear resistance. Liquefaction can lead to rapid settlement and deterioration of track geometry. While liquefaction primarily occurs in loose samples, cyclic mobility occurs in either loose or dense soils. Cyclic mobility tends to be driven by loading and has been associated with shear stress reversal (Selig and Chang 1981). Liquefaction and cyclic mobility may also occur in the subballast layer if the subballast has silt to fine sand particles, and the water in the subballast cannot drain adequately.

Progressive shear failure is the plastic flow of the soil caused by overstressing at the subgrade surface under repeated loading. The subgrade soil gradually squeezes out from under the tie and then upward at the tie end, following the path of least resistance as shown in Figure 3.20. This is primarily a problem with fine-grained soils, particularly those with high clay content. The addition of more ballast above the subgrade squeeze zone results in an increase in ballast depth and a corresponding tendency for a reduction in stress at the subgrade level, which might improve subgrade stability. However, the depression in the subgrade surface will trap water that can further soften the subgrade.

The formation of ballast pockets is an extreme case of cumulative plastic deformation. In the AREMA manual from 1921, the term water pocket is used to describe ballast pockets because the depression was filled with water. Although progressive shear failure is accompanied by progressive shear deformation near the subgrade surface, cumulative plastic deformation is classified as a separate type of subgrade problem because it includes not only the vertical component of progressive shear deformation.
but also the vertical deformation caused by progressive compaction and consolidation over a considerable depth of subgrade.

Frost heave occurs in the presence of frost-susceptible soil, free water, and freezing soil temperatures. Frost-susceptible soils are those that are sufficiently fine grained to have large capillary rise but are sufficiently pervious to allow an adequate flow of water to feed ice lens growth. These soils include silts, silty sands, and low-plasticity clays. Granular soils with very few fines are generally not frost susceptible. Frost action has two components: heave during freezing and softening during thawing. The frost heave develops from ice lenses that form within the soil, expand, and lift

Figure 3.20 Subgrade squeeze at end of tie: (a) track cross section with a subgrade squeeze and (b) subgrade squeeze and lime injection hole.
the ground surface. Rocks present near the surface will cool more rapidly than the adjacent ground, potentially causing an ice lens directly below the rock that can draw water from surrounding soil, grow, and force the rock upward. When the ice melts from the surface downward during thawing periods, excess water is released in the soil that causes a significant reduction of subgrade soil strength. During this period of thaw softening, the subgrade is susceptible to plastic deformation and shear failure, as described previously.

Volume change susceptible soils that shrink when dry and swell when wet move the track structure. Clay is the soil type predominantly susceptible to volume change associated with moisture content changes. These soils are mostly found in arid areas and contain large amounts of clay minerals. The volume change varies along the track and may lead to settlement (shrink) in some locations and heave (swell) in others, affecting track geometry. The largest volume change typically occurs on the initial swell of partially saturated soils.

Slope erosion occurs with running surface and subsurface water, but sometimes wind can also erode material at the subgrade slope and toe. Surface erosion may not immediately affect track operation. However, if not repaired and allowed to progress, erosion can undermine the track, lead to sinkholes, or otherwise impact track operation and stability. On an embankment, erosion may undermine the ballast shoulders and the ties. For a cut slope, this may lead to blocking drainage ditches, as eroded material flows down toward the track structure and may even be carried into the track structure in severe erosion situations.

Soil collapse is not common but can occur in soils that are most often windblown and deposited into a dispersed structure that is characterized by low compressibility when dry but suddenly softens when saturated. Loess deposits are notorious for this type of problem and, due to the stiff nature of the deposit, can often withstand very steep cut slopes, which are needed to reduce the soil’s exposure to water. Loess is typically windblown silt with some clay that binds the particles together in a dispersed and open structure.

3.3.4 Subgrade improvement methods

Subgrade problems vary widely, and each situation may require a unique solution. Subgrade repair and improvement is typically most successful if the problem is properly understood, and the devised solution addresses the problem. Properly understanding the problem involves understanding local geology, site history, previous construction methods and trends, local traffic trends, track maintenance history, track condition measurements, and site hydrology and hydrogeology followed by conducting a detailed site reconnaissance to identify the problem. Based on this level of understanding, a site investigation can be planned to further characterize the problem and design the repair. As subsequent sections of the book consider repair
and stabilization in detail along with specifics on site conditions, this section will focus on a few improvement methods that hold promise. Subgrade improvement techniques include grouting, load transfer, and soil densification-based techniques, and each of these will be discussed next.

Techniques of compaction grouting and displacement grouting tend to be practical options for existing lines, while permeation grouting tends not to be useful when the problem soil is fine grained and impermeable. Grouting techniques that rely on compaction or displacement for grout injection are likely to work better for repair of existing railroad subgrade problems. Compaction grouting relies on a stiff concrete mix that is compacted into a bulb in the ground compacting the neighboring soil and providing concrete structural support. Displacement grouting works in a similar manner, but the concrete is less stiff so it can be pumped under pressure while displacing adjacent soil. The grouting program should be designed to provide the needed confinement to stabilize the track structure. This can often be accomplished by identifying the path of least resistance for the failure, and reinforcing that section of the embankment so that other grouting locations do not accelerate the failure. Then, grouting in other planes can occur to retain the subgrade.

Load transfer involves installing a structural element in the track to transmit load through a problematic layer so that it reacts on a lower, more stable layer. Load transfer–based mechanisms include the compaction grouting method noted earlier, where the concrete column that remains is suitable for transferring load. Load transfer structural elements can be concrete or other cementitious material, steel, wood, and so on. Soil nails, concrete columns, and helical piles are likely candidates for load transfer. The main requirement is that the structural element be installed with the track structure in place, leading to the use of particularly small elements that fit between the ties.

Soil densification–based improvement methods include stone columns and other displacement-based techniques to compact the soil and fill the void with a structural element that can help transfer the load. Soil densification–based techniques, again like the compaction grouting method noted earlier, have the distinct advantage of compacting the surrounding soil during installation, resulting in improved soil behavior. Stone columns are another method for creating a structural element while compacting the surrounding soil and can be constructed by simply compacting stone in bulbs below the surface like the method of compaction grouting. Stone columns are trademarked as a rammed aggregate pier® or geopier®.

### 3.4 OTHER SUBSTRUCTURE MATERIALS

#### 3.4.1 Ballast treatment methods

A variety of treatment methods have been proposed for ballast over the years ranging from ballast glue and other pour-in-place material used to lock the ballast in place to the use of geosynthetics to provide tensile
reinforcement and geotextile fabric as a separator to keep fines in lower layers from migrating into the ballast. These materials are used to address the recognized need for ballast stability and confinement. However, claims regarding some of these products and treatments have not always been supported by evidence (Sussmann and Selig 1997). Therefore, the application of alternative ballast materials and stabilizing materials or additives should be viewed skeptically. For example, pouring anything into the void space of ballast that will block at least a portion of the voids will reduce the drainage and void storage capability of ballast at the least and in the worst case may attract and hold water.

The desire to stabilize ballast usually follows from the recognition that the ballast section is not properly compacted, because compact ballast is generally very stable. The main reason for poor ballast compaction is that ballast density is generally not controlled and compaction under traffic often accumulates slowly, even if aided by a dynamic track stabilizer following tamping or ballast placement.

Geotextiles have been proposed to provide separation from lower layers containing fine material that may contaminate the ballast. However, ballast has been found to abrade or puncture geotextiles, which removes any protection against separation and confinement (Figure 3.21). But even when geotextiles remain intact and do not puncture or tear, they often fail to prevent the upward migration of soils with an abundance of silt and clay particles from below, because such small particles readily pass through the geotextile and into the ballast. This indicates that the soil particles are finer than the holes in the geotextile, which should be designed using the filter separation criteria described in Chapter 6. Even if the geotextile does separate the fine material, problems can still develop as the geotextile may become caked with fine material and inhibit drainage. Lastly, consideration

![Figure 3.21 Abraded geotextile.](image-url)
should be given to how the geotextile may be removed during track maintenance or rehabilitation. Geotextiles have been particularly problematic because the material binds up on the undercutter chain, teeth, and sprockets.

The innovative use of new materials to improve track performance should be encouraged because the industry needs every economic advantage. However, healthy skepticism is required to ensure that the solution does not create problems beyond those being corrected. There are cases where installing geosynthetics and associated ballast treatments in track are appropriate; however, the application must fit within product capabilities and not inhibit future maintenance. It is prudent to place any geosynthetics below the maintained ballast layer at least 250 mm below the tie bottom, unless installing the material higher in the track structure will not inhibit track maintenance or ballast performance during or after its expected life. The material must not degrade ballast or substructure performance if destroyed by maintenance. For instance, some pour-in-place glues and binders confine ballast well, but if tamping is required and destroys the material matrix, the resulting small compressible (at least relative to hard rock ballast) pieces must not inhibit ballast interlocking, or create a track maintenance problem.

### 3.4.2 Alternative subballast materials

A variety of materials have been used in place of traditional granular sub-ballast. From concrete slab to asphalt layers, the intent is to place a material that is stiffer than subballast to provide improved track structure support. The track may already be underlain by old roadbed consisting of materials that provide good structural support. Sometimes this strong, stiff old road-bed layer is called hardpan, which is a very stable structural layer below the track that has compacted over time. In some cases, the hardpan is a result of decades of compaction that have produced a very dense layer that may be nearly as resistant to penetration as concrete. In other cases, this hardpan layer was intentionally placed, consisting of brick or cobble layers as an earlier attempt to support the track. In few instances, the placement of a layer of cementitious slag has been reported.

Alternative subballast materials that are sometimes considered for contemporary track designs include asphalt layers, reinforced soil layers, cement-stabilized layers, and impermeable membranes. Each of these alternatives has a variety of characteristics and requires knowledgeable designers. For example, asphalt can create a strong and stiff layer like a pavement or it can create a resilient layer for vibration attenuation, when properly specified and placed. Proper application of asphalt requires specific expertise in asphalt technology to ensure proper mix design, track section design, asphalt transportation, and placement techniques. Soil-reinforced layers can range from the use of high-strength geotextiles, to cellular geoweb, and to flat geogrid (Figure 3.22). The type of geotextile (woven or nonwoven)
will control the opening size that affects the filter characteristics, and also will affect the strength. Most geoweb is made from high-density polyethylene (HDPE), which is commonly used for plastic pipe, drainage infiltrators, and many other products. One problem with the use of HDPE in a structural layer is that HDPE tends to creep when loaded, which can result in strain over time, loss of structural support, and settlement. New materials are being used to limit the creep properties of HDPE geoweb, but these should be evaluated for durability and resistance to wear and abrasion.

Selecting an alternative subballast material may be a necessity when other solutions appear unlikely to create the required track structure and to adequately balance competing functional requirements. To ensure that alternative subballast materials are applied successfully requires that they address specific problematic soil conditions that the material is known to correct. Particularly, moisture-sensitive clay subgrade has successfully been treated using a glass fiber-reinforced geomembrane to seal the subballast and subgrade from water infiltration. Cement-stabilized soil layers have been used to successfully support track over soft subgrade. Soil reinforcement has been used to stabilize weak track structure on a steep embankment. These successes developed from understanding the problem and its failure mechanism, identifying a cost-effective solution, developing a conservative design, and following up to make sure the design is properly implemented.

### 3.4.3 Hot mix asphalt

The use of hot mix asphalt (HMA) as a ballast underlayment is the most common application of HMA in the track. As an underlayment, HMA is placed directly on top of the subgrade soil, or on top of a granular subbase, with ballast then placed on top of the HMA. HMA underlayment has been used in mainline track, road crossings, tunnel and bridge approaches, and
special trackwork such as turnouts and crossing diamonds (Chrismer et al. 1996). HMA has also been placed in the track as an overlayment with ties placed directly on the asphalt layer with no ballast. However, this application is not used in North American freight track applications due to the inability to surface and realign the track.

The main areas in which HMA underlayment can improve conventional track substructure performance are to provide drainage out of the track, reduce stresses on the subgrade, increase track stiffness, and improve constructability of new track.

Subgrade stress levels will be reduced beneath an HMA layer compared to an equal thickness of granular material (subballast) due to the increased resilient modulus that can be provided by HMA. This benefit is useful in situations where the thickness of the granular layer is limited due to overhead height or excavation depth restrictions that result from a variety of construction constraints. Without these restrictions, the desired subgrade stress reduction can often be accomplished with traditional granular materials.

The HMA also provides an improved degree of confinement to the track because HMA is much better than ballast at resisting spreading due to its higher resistance to tension and shear. The increased confinement will lead to a stiffer ballast layer response, if properly compacted. In some cases, the use of HMA can also result in track stiffness that is too high, causing accelerated rates of ballast deterioration and track component degradation (e.g., concrete tie cracking). The stiff HMA also typically results in less damping in the substructure (Singh et al. 2004) so dynamic loads may be more acute. HMA with a modified binder mix (e.g., rubberized HMA) can potentially improve the damping characteristics of the substructure to reduce high-impact stresses (Buonanno et al. 2000; Singh et al. 2004).

An HMA underlayment can also provide a smooth, rut-free surface for track laying operations, even in wet conditions, which can accelerate construction schedules. However, in locations with this problem, the construction of the HMA itself will need a reasonable work platform underneath it, requiring the placement of a granular working base in most cases.

It should not be assumed that traditional ballast and subballast materials cannot provide the advantages associated with HMA. For example, HMA may provide a stable work platform, but granular material placed over a soft soil can provide the same. Also, increased track stiffness can be provided by a stronger, stiffer, more confined ballast or subballast layer and does not require HMA. HMA has been applied at a number of locations throughout the industry partly due to widespread availability of both material and contractors who are knowledgeable in highway HMA applications. However, the use of HMA in track is different than in highway applications, and so careful consideration must be given to the design details, contractor selection, and material specifications.

HMA design and property considerations involve two main areas of concern: mix design/quality control and structural design of the track cross section.
Mix design is covered thoroughly in many textbooks, design manuals, and industry standards (e.g., Huang 1991; Asphalt manual 2012). These references describe the process of developing a mix design from desired properties through proper selection of aggregate and amount and viscosity of asphalt binder. Applications of HMA in track have ranged from providing an alternative structural layer to sealing the surface of problematic subgrade. However, the use of asphalt to seal water from penetrating into poorly performing subgrade cannot be successful if the layer is not properly designed for structural performance. In almost any HMA track installation, asphalt layer strength will be a primary concern, which will require use of a large angular aggregate relative to the common highway gradations. From that point forward, the mix design will vary depending on the desired characteristics including resilience, strength, and deformation properties. See Chapter 5 for discussion of design trade-offs and considerations as well as references on various designs for track installation.

Any installation of asphalt should include basic quality control parameters to ensure conformance with design specifications. Temperature, density, and strength testing should be required for any installation, in addition to other required quality control parameters specific to the design or application. The logistics of trucking the material and maintaining the asphalt temperature is a concern due to the remoteness of many construction sites. Achieving the desired compacted density can be somewhat challenging to a contractor not familiar with railroad HMA applications because compaction is often difficult on track, especially over a soft subgrade. Control of density is essential because it will control the strength of the layer, and strength is critical to long-term performance. Finally, samples from the placed layer should be obtained, tested, and compared with the mix design and specification as well as the expectations of the track design engineer.

Often the most difficult information to estimate is the performance of HMA in distributing applied stress and resisting deformation. Use of a geotechnical track analysis software programs like KENTRACK or GEOTRACK is the main tool available for this analysis. The only way to verify the design from these types of analysis is to obtain field samples to test for resilient modulus and other mechanistic design parameters and then conduct a field test to evaluate the deformation of each individual layer, or the overall track system. More detailed design considerations and material properties are discussed in Chapter 5.

### 3.4.4 Concrete and cementitious material

The use of concrete in track has a long history from use of concrete beams to support the rails (Chipman 1930) to the use of contemporary concrete cross-ties. Along the way, major problems with the widespread use of concrete have been encountered including alkali-silica-related concrete tie failures of the 1980s. Continued vigilance of the concrete product quality is
required to ensure that track integrity and stability are not compromised. All concrete quality control, design, and maintenance-related specifications should be heeded.

In addition to the requirement that good-quality concrete be used in track, the track structure must be stable for the use of any concrete structure (Li et al. 2010). Concrete requires solid support, whether for a concrete tie or slab. Concrete is unforgiving of poor support conditions, and excessive settlement could lead to cracks and rapid structural deterioration. One well-documented case of advanced track design and failure of concrete ties and a cement-stabilized subgrade layer is the Kansas Test Track (Cooper et al. 1979).

The Kansas Test Track was a 1.5-mile-long experiment in the design and construction of advanced track technology. A wide variety of advanced track support structures were installed to provide different levels of track stiffness in the test zone with a plan for evaluation over multiple years. Signs of distress became apparent after only 2.5 months of operation and the track was closed to traffic permanently within 6 months. The poor track performance resulted from poor drainage of the top of the embankment onto which these various track support structures were installed. This poor drainage resulted in softening of the embankment, which compromised the performance of the track support structures (Cooper et al. 1979). This highlights the importance of the drainage and geotechnical aspects of track support, especially for concrete structures that cannot withstand large track deformations without cracking and structural deterioration.

### 3.4.5 Water

The water trapped in and draining through track is a drainage concern as already described, especially considering the potential effects on subballast and subgrade performance. In many locations at a depth of perhaps only one or two ballast particles below the surface, moisture can be noted that is retained in the track from previous wetting. Deeper, perhaps near the bottom of tie, the presence of water is common when ballast is fouled, except in the most arid of conditions. Water is often a part of the track, and even in well-drained situations, inundation during a rainfall event is likely to cause water to be carried through the track structure.

Although water is typically thought to be chemically neutral, it may have high or low pH and/or carry chemical and organics that can cause damage. Water is a solvent that tends to take soluble elements, organics, and chemicals into solution and transport them along with the soil particles as described in the separation/filtration discussion of the subballast section. Salt is an element that most readily dissolves and is abundant in many areas of the world. Drainage water can carry dissolved salts and sulfates that can be detrimental to various track components and affect grounding of track signals or development of stray currents. Salts attack steel and metals and
cause rust through oxidation-based corrosion. Sulfates attack concrete elements or anything containing cement, including drainage pipe. Precautions to deal with these potential problems include special coatings for steel or cast iron pipe and use of chemical resistant concrete. To anticipate and manage these problems one must evaluate the site conditions to determine their likelihood, understand the site-specific nature of this problem, and recognize signs indicating that it could or may be occurring, and apply the available means to deal with these conditions.

3.5 TRACK TRANSITIONS

A track transition is a change in track structure, such as encountered by a vehicle moving from a conventional ballasted track on a soil subgrade to a bridge, slab track, or grade crossing. A transition can either involve a change of track superstructure (such as at grade crossing and turnout) or a change in track support (bridge, slab, tunnel, etc.). In most cases, a track transition involves changes in both track superstructure as well as track substructure. Mechanistically, a transition is often characterized as a large change in vertical track stiffness, although this is not always the case. The problems at transitions vary, but the most common feature is differential track settlement, such as a dip in the vertical rail profile shown in Figure 3.23.

Typical problems associated with transitions and the settlement-induced dips are rapid track geometry degradation, mud pumping, poor ride quality, difficulty in conducting effective track maintenance, and damage to track and vehicle components from dynamic wheel loads. There are also problems transitioning from low track stiffness to high track stiffness, such as when

Figure 3.23 Dip at bridge transition.
trains travel onto a bridge where the common problems include excessive vibration and breakage of track components on the bridge.

A site investigation is often required to determine if the recurring dip that follows maintenance is related to improper maintenance or from instability of the track structure or substructure. Maintenance procedures that work well in open track away from the transition may require modification to be effective in the transition. Many misconceptions exist about the source of these problems and their appropriate repair. A full discussion of track transition–related problems, causes, and potential remedial measures is provided in Chapters 5 and 9.