Parametric Analysis of the Stability of Degraded Railroad Platforms

Paulo Roberto Aguiar and Cassio Eduardo Lima de Paiva

Geotechnical and Transportation Department, Civil Engineering School, Campinas University (UNICAMP), Campinas 13000-000, Brazil

Abstract: The current study is developed based on one of the types of subgrade rupture risk suggested by Selig and Waters (1994). It aims to evaluate subgrade stability railroad, which consists of the slope stability analysis of a railway yard embankment subjected to a wagon load type gondola parked on this track. This proposed analysis was taken into consideration because there are Brazilian railroads in high deterioration level. In some of these lines the tracks are submitted to increasing amount of load every year. The adopted model slope stability to the studied railroad embankments considers the rupture on circular line. It was applied the Geoslope-Slope/W software, version 6, to the evaluation of the platform-slope system. Several situations are adopted to reach the minimum safety slope, permitting to analyze the platform stability to keep railroad traffic under adequate safety level.

Key words: Rupture circle, railway yard track, tensions at subgrade, degraded railway track.

1. Introduction

Most of Brazilian rail network was built until the 1950s. The use of sub-ballast layer between ballast and platform was not common in these lines. Because of this, ballast contamination was frequently observed. Brazilian railroads were franchised during these 20 years. The dealers were specialized on load carrying. After the Brazilian railroad privatization program, dealers started to focus its financial resources mainly to renew rolling stock. As to main operational aspect, the dealers have been increasing successfully the amount of carried loads and, therefore, the payback has been acceptable. However, these companies have to deal with a paradox: how it can increase carried loads amount, using expensive vehicles on railroad under increasing damage level and trains running faster under reduced safety levels?

The objective of this study, detailed by Aguiar [1] was to evaluate the Brazilian rail network, from railroad parts sample, mainly degraded ones, following an unusual criterion. This criterion takes into consideration cross section, platform and nearby slopes features under the risk of failure on circle line rupture. This method is currently used for a new railroad design, not for maintenance procedure. No technical references were found for this. This evaluation, suggested by Selig and Waters [2], considers rupture risk associated to the degraded railroad, as the evaluation of efficiency loss on drainage devices or ballast contamination due to the carried particles of soft clay. In this case, the studied slope loses its original shape, and, therefore, its stability. Currently, the increase of the Brazilian carried load on the scenario of the ongoing superstructure deterioration justifies the necessity of the platform inclusion on the circle shape failure model. This way of evaluation is mandatory in developing countries, mainly railroad network working under continuous deterioration.

The main objective of this study is to evaluate the rupture of the platform, from the top up to 1.0 m depth, due to geotechnical degradation. It is highlighted that it is common to enhance older Brazilian railroads with a layer of clean ballast on old and degraded ballast,
instead of the correct service of replacing or cleaning the ballast, as suggested by Indraratna et al. [3]. This feature is considered to justify the main objective of this work.

The adopted failure model, the circle type applied to the studied platform, which has track and railroad layers of the superstructure to search the circle of highest risk. It is also considered the undercarriage effect, spreading stresses to the track, sleepers and to the platform, analyzed until the limits of the embankment, i.e., the considered slope. The undercarriage studied is a gondola for loads, parked on the nearest track to the studied slope. This system is uploaded to the GEOSLOPE software—Slope/W version 6, that calculates the safety factor to any situation established, i.e., any failure circle studied.

2. The Developed Study

This study allows to solve this main question:

What is the critical rupture circle failure to a low bearing capacity railroad platform submitted to static load from a loaded parked car on the track, ballast and subgrade saturated and contaminated system and deficient drainage?

3. Materials and Methods

It was conceived a group of parameters to approach as much as possible the real scenario of a typical Brazilian railroad: overcharged, damaged and worn out. There were selected railroads in precarious conditions, as representative ones to the current Brazilian railroads. It was also adopted a static condition of load action on the railroad to the track model. The platform is considered saturation, because of the high water table next to a small slope of an embankment. The surface platform layer named SPL (surface platform layer) is saturated and it is composed by mixed soil and ballast. In order to evaluate this system, it was defined a cross section of study, according to a track next to a slope of an embankment, without a sub-surface drainage device, shown at Fig. 1. According to Fig. 1, the thickness of the contaminated ballast layer is 0.10 m.

The chosen load concerns to a parked gondola car, weighing up to 1,176 kN (120,000 kg) and composed by four axles, separated by 1.75 m. The distance from the first axle to the coupling is 1.58 m. The considered load is composed of four axles from an adopted rear bogie wagon and the front bogie of the next one. The distances between them are: 1.75 m between first and second axles, 1.58 \times 2 = 3.16 m between second and third ones (near the wagons coupling) and 1.75 m between third and fourth ones.

There are also adopted:

Gauge: BG (broad gauge) \( b = 1.60 \) m, NG (narrow gauge), \( b = 1.00 \) m and DG (double gauge) composed by broad and narrow gauges;

Wooden sleepers: 2.80 m length (L), 0.24 m width (W), 0.17 m height (H) applied for double and broad

![Fig. 1 Adopted railroad cross section. Legend: BT—ballast, CB—contaminated ballast, SPL—surface platform layer, AC—compacted fill.](image-url)
Parametric Analysis of the Stability of Degraded Railroad Platforms

gauges. Narrow gauge sleepers: 2.30 m (L), 0.22 m (W), 0.16 m (H); Studied rails: 115 RE AREMA (American Railway Engineering and Maintenance of Way Association) of \( I = 2,370.5 \text{ cm}^4 \), when it is new. After worn out, near \( I = 2,255.85 \text{ cm}^4 \), 25% of wear out considered. To the 100 RE rails (AREMA pattern), under wear out condition: \( I = 1,695.20 \text{ cm}^4 \).

SPL: a superficial and saturated layer composed by soil and ballast, of 0.30 m to 1.10 m width.

Ballast layers thicknesses: \( h \) equals to 0.15 m and 0.35 m. In both cases, the first 0.10 m are composed by contaminated ballast under elastic module \( (E) \) of 80 MPa (800 kgf/cm²). The remaining thickness works under elastic module \( (E) \) of 150 MPa (1,500 kgf/cm²).

The studied platform embankment slopes steepnesses are: 1.5:1 (1:0.66); 1:1; 1:1.5 and 1:2; respectively applied to horizontal and vertical relationship.

Heights of the embankment \( (H) \): 0.25, 0.50, 0.75, 1.0, 2.0, 3.0 and 5.0 m.

Distance between the ballast toe and the edge of embankment \( (d) \): 0, 0.10, 0.20, 0.30 and 0.60 m.

Average sleeper spacing: 0.60 m. Nevertheless, it was also studied a case where the spacing ranged from 0.55 m to 0.65 m.

Compression stress on the platform ranges from 0 to 280 kPa, amplitude from an adopted stress distribution model applied to the platform.

Winkler coefficients \( (c) \): 0.02-0.14 N/mm³ (2-14 kgf/cm³).

Soil features of the studied layer:
- SPL layer: bulk density 16, 18 and 20 kN/m³; cohesion resistance: 5, 10 and 15 kPa; internal friction angle: 0 to 25°;
- Platform soil (compacted fill): density: 16, 18 and 20 kN/m³; cohesion resistance: 5, 10, 15, 20, 30 and 60 kPa; internal friction angle: 0 to 30°.

Tamping ballast width (under sleepers): BG (broad gauge): 1.13 m; NG (narrow gauge): 0.76 m; DG (double gauge): 0.80 and 1.40 m.

The study was developed according to the following steps and conditions:
- determination of the load under the sleeper from the load of the adopted truck using the Zimmermann and Carothers-Terzaghi equations;
- determination of the stresses at the sleepers and ballast interface, according to each studied gauge;
- determination of the stresses at the platform due to the central sleeper D1 and to the adjacent ones (D2, D3, D4 and D5) according to Fig. 2;

![Fig. 2](https://via.placeholder.com/150)

**Fig. 2** Strain diagram of wheel on rail 1, spread to sleepers D2, D3, D4 and D5—measurements on central sleeper axis: (a) plant; (b) railroad longitudinal view.
determination of the isobar lines under the platform, i.e., the stresses distribution under the platform;

- entry of the isobars and platform system to the software Geoslope;
- research of failure circles;
- analysis of results.

The load on the platform which was charged to the used software generated vertical stresses on the platform. The graph of these stresses shows a double bell shape, according to Fig. 3.

Once obtained the ballast and platform system isobars, the software Geoslope permitted to achieve the safety factor according to Fig. 3, that shows:

- The obtained failure circle crosses the edges of segments AB and CD. The rails and sleeper system is inside the soil mass under rupture, i.e., outside the B point;

- The layer 1 refers to the SPL, considering its thickness, specific weight, internal friction angle and cohesion;

- The layer 2 refers to the embankment soil, considering specific weight; internal friction angle and cohesion;

- Physical dimensions of the slope, steepness and height.

There were obtained the FS (safety factors) from Morgenstern and Prince Method, according to Krahn [4], who considers the force momentum balanced between horizontal forces of shear strength of soil and soil effort and from the equilibrium of the horizontal forces.

4. Results

There were performed more than 2,300 experiences to determine circular rupture safety, considering: strain platform, embankment height, horizontal projection and steepness of the embankment to 3 gauges. There were also developed experiences to evaluate the soil stability and of the SPL. Other considered feature: rail wear out level and embankment height.

The goal of these experiments was to analyze each variable in an isolated way, evaluating its influence on the obtained results and if each variable generated effective variation to the safety factor. There were studied thirteen variables.

The most important achieved variables were: steepness of the slope, gauge, height of slope, soil cohesion and internal friction of angle from platform soil and the internal friction of angle of the SPL layer.

The following variables showed low sensitivity or no influence at the results: distance “d” between the ballast toe and the slope crest; cohesion of the SPL; thickness of the SPL layer; thickness of the ballast layer; kind of rail and head wear; specific weight of the SPL and the specific weight of the platform soil.

Fig. 3 Typical cross section used to evaluate the circle failure.
4.1 Slope Steepness

The FS and slope steepness correlation is shown in Fig. 3. The safety factor FS is related to the failure circle as a function of the slope steepness (in radians). The dashed line (FS = 1.1) is the stability limit, adopted in this work. FS ranges from 1.3 to 1.5, commonly, in this study it is considered 1.1 as the lowest value, because this studied case is not applied to Brazilian railways.

The continuous line of Fig. 4 crosses the adopted limit of factor (FS = 1.1) correlated to one radian slope steepness. In this figure, steepness slopes of an angle higher than 1 radian are correlated to unstable situation. This continuous line is not a straight one. Then, the FS = 2.02 was obtained, correlated to the initial point of abscissa of Fig. 4.

4.2 Gauge

The results of this study are shown in Table 1. The heights of embankment \( H \) are on the left column. The relationship between the FS obtained for the gauge analyzed, the narrow one, and the FS obtained for the broad gauge is shown as the \( G \) values columns.

\[
G_1 = \frac{FS\, (NG)}{FS\, (BL)} \quad (1)
\]

\[
G_2 = \frac{FS\, (DG)}{FS\, (BL)} \quad (2)
\]

FS values which are under 1.1 establish risky condition. The narrow gauge FS values are 9% to 21% lower than broad gauge case, for \( H = 1.0 \) and \( H = 0.5 \) m heights of the embankment. The FS from narrow gauges studies are 15% lower than broad gauge, applied to embankment of more than 1.0 m height. The FS values practically do not vary between double and single lines, applied to broad gauge. Its variation is around 3%. The gauge width generates significant result variation, assuring this factor is an important one.

It is mandatory to reduce the platform stresses from 280 kPa to an interval of 135 kPa to 200 kPa, in order to achieve a FS equivalency for narrow and broad gauge, according to the height of the embankment. This condition corresponds to a load reduction of 48% to 28% for each wheel. It is also equivalent to a decrease of 147 kN (15,000 kg) of a full load truck or a wheel load decrease interval of 70.5-105.9 kN (7,200-10,800 kg). Considering the studied loaded gondola car, its total

![Fig. 4 Variation of FS as a result of the angle of the slope.](image)

<table>
<thead>
<tr>
<th>Height of embankment (m)</th>
<th>FS values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Broad gauge</td>
<td>Narrow gauge (NG)</td>
</tr>
<tr>
<td>1.0</td>
<td>1.52</td>
</tr>
<tr>
<td>5.0</td>
<td>0.85</td>
</tr>
</tbody>
</table>
weight will be reduced from 1,176 kN (120,000 kg) to a value of approximately 558 kN to 843 kN (5,700 kg to 8,600 kg).

To counterbalance railway operational risk between tracks from 1.6 m to 1.0 m gauges, it is necessary to reduce stresses on the top of the embankment, from 280 kPa to 135-200 kPa interval. The selected value depends on the embankment height. This means a reduction of 48% to 28% at the considered wheel load.

4.3 Height of the Slope

The results are shown in Table 2. The bold FS values refer to the situations where there is a slope risk of failure, i.e., FS ≤ 1.1.

According to this software, slopes more than 2.0 m height are submitted to higher risk failure. This case is applied to unusual slopes 1.5 to 1 steepness in worldwide sense but are, very common in old Brazilian railroads. The same values of Table 2 are shown in Fig. 5, where it is shown a graph of height of the embankment per FS values. The dashed line limits the adopted 1.1 factor of safety. The values on the left of 2.0 m height of embankment are related to safer condition, higher than FS = 1.1, material homogeneity near the slope and the figures for great representation of the SPL explain these higher values. There is an inverse and significant relationship between height of the slope and FS, according to the Eq. (1) (R² = correlation coefficient = 0.978).

\[ FS = 1.4001 \times H^{-0.3767} \]  

4.4 Cohesion of Platform Soil

The cohesion of platform soil varies from 5 kPa to 60 kPa. Fig. 6 shows results for FS in function of SPL cohesion. The dashed line which refers to 1.0 m height slope shows safer values for cohesion values higher than 10 kPa. The dotted line which refers to 5.0 m height slope shows safer values for cohesion values higher than 45 kPa. The straight line refers to the FS = 1.1, the adopted stability.

The relationship between “cohesion of the platform soil” and FS features a polynomial line to 1.0 m height slope and a linear feature to 5.0 m height slope.

Table 2  Values of the FS coefficient as a result of the height of the embankment (H).

<table>
<thead>
<tr>
<th>Height of slope (m)</th>
<th>FS values</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td>2.09</td>
</tr>
<tr>
<td>0.50</td>
<td>2.09</td>
</tr>
<tr>
<td>0.75</td>
<td>1.64</td>
</tr>
<tr>
<td>1.00</td>
<td>1.47</td>
</tr>
<tr>
<td>2.00</td>
<td>1.07</td>
</tr>
<tr>
<td>3.00</td>
<td>0.88</td>
</tr>
<tr>
<td>5.00</td>
<td>0.73</td>
</tr>
<tr>
<td>10.00</td>
<td>0.62</td>
</tr>
</tbody>
</table>

Fig. 5  Variation of FS per height of the embankment.
Fig. 6  Graph of the variation of FS × cohesion of platform soil.

Table 3  Values of FS for the angle of friction of the platform soil (AC) × height of the slope.

<table>
<thead>
<tr>
<th>Height of the slope embankment (m)</th>
<th>Angle of friction of platform soil (degrees)</th>
<th>FS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>0</td>
<td>0.36</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>0.61</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>1.09</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>1.32</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>1.47</td>
</tr>
<tr>
<td>5.0</td>
<td>0</td>
<td>0.18</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>0.30</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>0.41</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>0.52</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>0.62</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>0.73</td>
</tr>
</tbody>
</table>

Table 4  Values of FS for the angle of friction of the SPL according to height of the slope.

<table>
<thead>
<tr>
<th>Height of the slope embankment (m)</th>
<th>Angle of friction of SPL (degrees)</th>
<th>FS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>0</td>
<td>0.22</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>0.55</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>0.88</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>1.18</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>1.43</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>1.70</td>
</tr>
<tr>
<td>5.0</td>
<td>0</td>
<td>0.20</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>0.50</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>0.67</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>0.70</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>0.73</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>0.76</td>
</tr>
</tbody>
</table>

4.5 Platform Soil (AC) Angle of Friction

It was studied a 0° to 25° degrees amplitude of angle of friction to the platform soil (AC). Table 3 shows angle of friction and height of embankment values related to FS values.

Table 3 shows stability just for 1.0 m height of slope related to SPL angle of friction of 20° or 25°.

4.6 SPL Angle of Friction

The studied SPL angle of friction varies from 0° to 25°. The results obtained for FS × angle of friction of the SPL layer are shown in Table 4. The left column shows the experimental values for the height of the slope. At the central column they are the values of angle of friction of the SPL layer, and the right column shows the results obtained for the FS factor.

It is possible to observe, in Table 4, that only two of all the experiments done resulted in slope stability. The sensitivity of this variable is significant and directly proportional to FS and its variation is linear when the height of the slope is \( H = 1.0 \) m, and a third order polynomial when \( H = 5.0 \) m.

5. Conclusions

These analyses permitted to conclude that angle and height of slope, SPL angle of friction, platform friction
angle and cohesion, kind of gauge influence the slope of embankment FS, under each studied circle of failure.

Under higher stability risk, the critical approach is related to:

1. Broad gauge (BL) (maximum stress at the platform: 210 kPa):
   - Slope 1.5:1—$0 \leq d \leq 0.20$ m, $H \geq 1.0$ m; $d > 0.20$ m; $H \geq 2.0$ m;
   - Slope 1:1—$d = 0$; $H > 2.0$ m; $d \geq 0.10$ m; $H > 2.0$ m;
   - Slope 1:1.5—$0 \leq d \leq 0.10$ m; $H > 2.0$ m; $d > 0.20$ m; $H \geq 3.0$ m; for angle of friction below 15° (SPL layer), and $d = 0$; for angle of friction below 25° (SPL layer), and $d = 0$;
   - Slope 1:2.0—There is no critical situation.

2. Narrow gauge (NG) (maximum stress at the platform: 280 kPa):
   - In every slope evaluated, to any value of “$d$”, and $H > 1.0$ m;
   - In case the 25% stress reduction necessity is respected to the narrow gauge: Slope 1.5:1 and Slope 1:1 are always critical; Slope 1:1.5 = $d = 0$, $H > 1.0$ m; $0.10 \leq d \leq 0.30$ m, $H > 2.0$ m; $d > 0.30$ m, $H \geq 3.0$ m.

3. Double gauge (DG) (maximum tension at the platform: 185 kPa):
   - Slope 1.5:1—$0 \leq d \leq 0.10$ m, $H \geq 2.0$ m; $d > 0.10$ m; $H > 2.0$ m;
   - Slope 1:1—$0 \leq d \leq 0.60$ m; $H > 2.0$ m; $d \geq 0.60$ m; $H > 3.0$ m;
   - Slope 1:1.5—$0 \leq d \leq 0.60$ m; $H \geq 5.0$ m;
   - Slope 1:2.0—No critical situation.

There was no risk failure to a flat platform, broad gauge submitted, distinguished to Selig and Waters [2] values. It happened likely to the use of a constant load equivalent to the maximum tension at the platform. This tension is distributed on the railroad cross section.

Although the curves of stress distribution to the broad and double gauges were different, due to the differences of ballast tamping, which causes different stresses on the platform, the final results were very similar.

With the obtained results it is possible to conclude that there is higher stability risk of the railroad platform near the embankment slope. As this is not a deep study it is suggested to be better analyzed under a greater amount of results to conclude a specific new case of railroad stability. This work studied just a SPL under a degraded surface under it. On the other hand, this study permitted to investigate and to analyze railroad conditions under the circular line rupture applied to the system platform-slope, by Selig and Waters [2] as a distinguished way of maintenance management.

References


