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# Design methods for geogrid stabilization of working platforms

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**Abstract.** Working platforms are temporary structures that provide a suitable foundation for heavy construction plant, working machines (e.g. piling rig and cranes) and temporary construction elements (e.g. temporary lifting bridge or staging areas). Working platforms can be greatly improved by geosynthetics stabilization. This paper describes the design methods for geogrid stabilization of working platforms, which have a critical safety role in the construction industry. A successful case history is presented.

## 1 Introduction

The design of granular working platform for heavy construction plant and working machines traditionally has been carried out using "empirical" methods, based on suitable materials and platform thickness used in previous projects with similar loadings and ground conditions. This empirical approach has, on occasions, resulted in catastrophic failure and significant incidents.

This paper aims to present the recent and current design methods for the design of granular working platforms, which are based on theoretical models that are proved to be ideal and reliable for geosynthetics stabilized soil platforms on soft clay subgrade. Those models are usually characterized by the following geotechnical parameters in undrained condition: $\varphi = \varphi_u = 0$ and $c = c_u$.

## 2 Traditional design approaches

The publication of CIRIA SP123 [1] and BRE BR470 (issued back in 2004 and reviewed in 2011 [2]) in the UK introduced analytical design approaches for the design of both unstabilized and geosynthetics stabilized granular working platforms.

On the BS8004:2015 standard [3] it is noted that "geosynthetics incorporated into the construction of granular working platforms might provide beneficial effects that enhance the stability of the working platform". It is also noted that when a geosynthetics material is introduced within the granular platform it will be necessary to use an alternative design method, and it is recommended that this design is undertaken with the support of a geosynthetics manufacturer.

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CIRIA SP123 [1] provides guidance on the use of geosynthetic reinforcement in various soil structures and applications. Considering working platforms, its approach is based on classical bearing capacity methods with an allowance for lateral stresses in the platform material. The method uses partial factor with ULS checks (bearing capacity and geosynthetic reinforcement strength) and SLS checks on geosynthetics reinforcement. The complexity of the calculations and the limitation in terms of load spread angle, partial factors for geosynthetics reinforcement and the choice of a single strata sub-formation design, represent important limits of this method.

On the other hand, the BRE BR470 [2] is based on classical bearing capacity methods but uses the concept of punching shear capacity within the platform as suggested by the experimental model developed by Meyerhof. Instead of assuming a load spread through the platform, it is assumed that punching shear resistance develops within the platform thus partially supporting the applied load and reducing bearing pressures on the formation. Checks on bearing capacity are deemed to satisfy limits on settlement. This is not a limit state method, and the factors should not be viewed as partial factors. No strength factors are applied to the formation or fill but a factor of 2 is applied to geosynthetic reinforcement strength, to limit deformation under load to an acceptable amount. Unlike the CIRIA SP123 model, in the BRE BR470 geosynthetic reinforcement is not considered to provide lateral restraint. Instead, it is considered to provide additional vertical restraint at the punching perimeter, which further reduces the bearing pressure on the formation. Both guidelines give instruction for single layers of reinforcement placed at the formation of working platforms and both ignore the benefit provided by any other reinforcing layers. The BR470 method of analysis is only representative if punching type failure occurs through the platform and in the subgrade; but for most subgrades this is not considered to be representative of the actual failure mode. BRE BR470 recognised that alternative design methods may be used for geosynthetics in situations for which they have been validated.

## 3 Static method for rectangular loaded area

Referring to previous research [4], the Static method, which is applicable for designing working platform over soft clay soil, assumes that the bearing capacity of an unstabilized platform on a soft clay sub-grade is:

$$q_u = \pi\, c_u \tag{1}$$

and assumes that the bearing capacity of a geosynthetics stabilized platform on a soft clay subgrade is defined as:

$$q_s = 2\, \pi\, c_u \tag{2}$$

where $c_u$ = undrained shear strength.

The Static design method is based on the distribution of vehicle track pressures throughout a geosynthetic stabilized layer and it ensures that the pressure at the top of subgrade is less than the allowable bearing pressure of the subgrade soil, divided by a defined Factor of Safety. The Static Method assumes that the vertical pressures are distributed through the platform soil layer according to Boussinesq theory [5] for uniform load on a rectangular loaded area: hence the width and length of the crawler track will be considered as loaded area, as shown in Fig. 1. The Boussinesq equation provides the induced vertical stress at any point below the rectangular loaded area.

An empirical relation between CBR value and the undrained shear strength can be used if subgrade CBR value is provided:

$$c_u = 30\ \text{CBR}\ \ (\text{kPa}) \tag{3}$$

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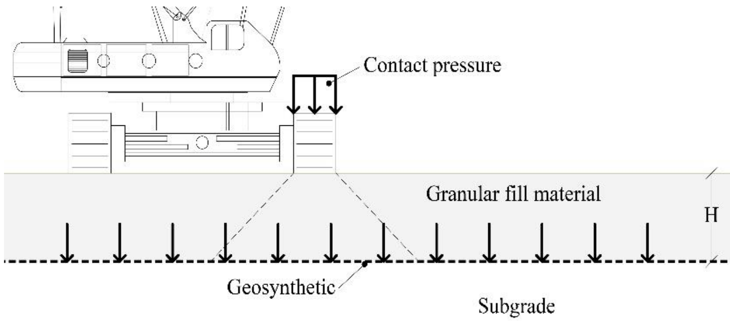


**Fig. 1**. The Static method considers uniform load on a rectangular loaded area (from [4])

As shown in Fig. 2, if a vertical pressure q is applied, evenly distributed on a rectangular area, to a homogeneous elastic half space, since the overburden stress is constant, the application of the pressure q generates equilibrium conditions in every point of the medium. Hence the vertical stress is independent from the medium characteristics. The Boussinesq theory provides the stress components in every point along the vertical line passing by one corner of the rectangle [5].

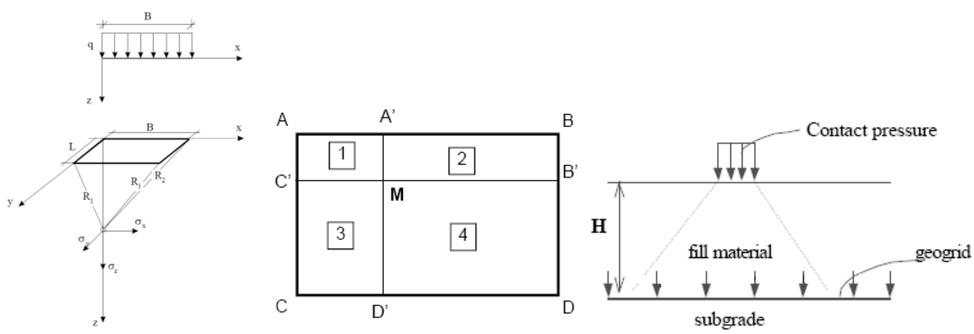


**Figure 2.** Scheme for Boussinesq Equation (left), scheme for calculating the stress along the vertical passing by the point M (center), scheme for bearing capacity calculation (right)

With reference to Fig. 2, left, the following equations apply:

$$R_1 = \left(L^2 + z^2\right)^{0,5}$$
$$R_2 = \left(B^2 + z^2\right)^{0,5}$$
$$R_3 = \left(L^2 + B^2 + z^2\right)^{0,5} \quad (4, 5, 6)$$

$$\sigma_z = \frac{q}{2 \cdot \pi} \cdot \left[ \arctan\left(\frac{L \cdot B}{z \cdot R_3}\right) + \frac{L \cdot B \cdot z}{R_3} \cdot \left(\frac{1}{R_1^2} + \frac{1}{R_2^2}\right)\right] \quad (7)$$

where:
$\sigma_z$ = induced vertical stress at depth z (kPa)
q = uniform pressure on the rectangular area (kPa)
z = depth below surface (m)
L, B = length and width of the loaded rectangle (m)

It is possible to calculate the stress along a vertical line from a point inside the rectangular loaded area by dividing the whole area into 4 rectangles (Fig. 2, center). By applying the principle of superposition, the total stress at a point will be the sum of the stresses generated by the rectangles 1, 2, 3, 4.

If the platform is unstabilized, local over-stressing in shear takes place before a complete failure of soft soil subgrade occurs, resulting in punching shear failure or local shear failure in the soil. Under such conditions the bearing capacity of the subgrade (Fig. 2, right) can be calculated by Eq. (1). When localized shear failure of the subgrade can be prevented (i.e., a general shear failure can be reached), that is when the platform is stabilized with geosynthetics, the bearing capacity of the subgrade can be calculated by Eq. (2).

Rutting and deformations of the platform can be limited by reducing the allowable bearing capacity through an appropriate Factor of Safety FS: a value FS = 2 already affords good reduction of deformations and displacements; a value FS = 3 can be selected when allowable deformations are minimal:

$$q_{ua} = q_u \ / \ FS \tag{8}$$

$$q_{sa} = q_s \ / \ FS \tag{9}$$

Therefore, the thickness of the platform, usually made up of granular soil with good frictional characteristics, is equal to the depth z at which the vertical stress calculated by Eq. (7) becomes equal to $q_{ua}$ for the unstabilized platform and to $q_{sa}$ for the stabilized platform.

The required foundation thickness (h) can be plotted versus the undrained cohesion ($C_u$) of the subgrade for both stabilized and unstabilized soil (Fig. 3). If a geogrid is introduced in the system, the thickness of the foundation decreases dramatically.
More details about the Static method can be found in [4, 5, 6, 7, 8].

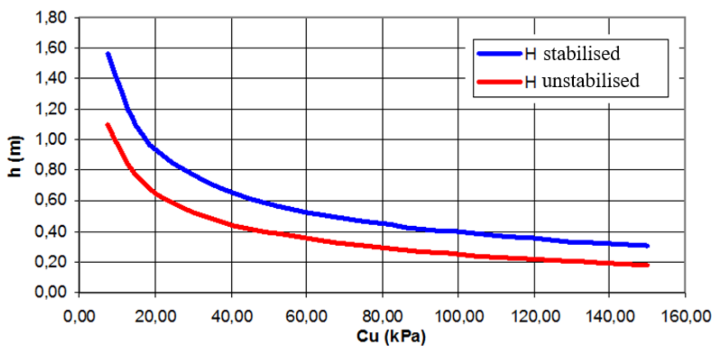


**Figure 3.** Typical plot of the required platform thickness H vs the undrained cohesion $c_u$ of the subgrade considering a stabilized (red line) and unstabilized (blue line) foundation

## 4 T-value method

Lees [9] developed non-dimensional relationships between the bearing capacity ratio $q_u$ / $q_s$ and the load transfer efficiency of the granular layer expressed as a dimensionless T value:

$$\frac{q_u}{q_s} = 1 + T\,\frac{H}{B} \leq \frac{q_g}{q_s} \quad \text{(strip footing)} \tag{10}$$

$$\frac{q_u}{q_s} = \left(1 + T\,\frac{H}{B}\right)^2 \leq \frac{q_g}{q_s} \quad \text{(square or circular footing)} \tag{11}$$

The T value depends on the shear strengths of the two layers and it is derived by a parametric study with numerical analyses, and by physical testing, the results of which are shown as the lower unstabilized curves in Fig. 4, where $S_u$ is the undrained shear strength of clay and $P'_0$ is the effective vertical stress at base of granular layer. These formulas and chart allows a simple calculation of the bearing capacity directly from the shear strengths of the individual soil layers. It can be applied to both surface and shallow embedded foundations, circular or square or strip footing, and with dry or saturated granular layers. The bearing capacity of foundations with B/L ratios between 0 and 1 can be determined by linear interpolation.

It has to be noted that the track of a crawler crane is neither a strip footing of practically infinite length (for which the length L can be disregarded) nor a square or circular footing.

Therefore, in case of a working platform loaded by a crawler crane or similar tracked machine, the T-value method may produce just an approximate solution.

Moreover, comparing Fig. 4 with Fig. 3, it seems that the T-value method, although developed for a specific type of geogrid, would provide similar results to the Static method.

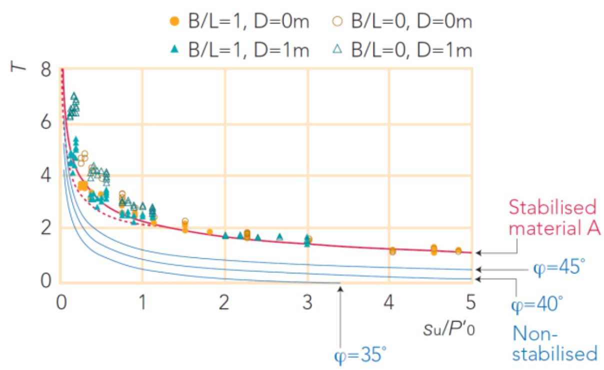


**Fig. 4.** Variation of T with $S_u$ for specific geogrid product and aggregate (from [9])

## 5 Geogrid design

Geosynthetics afford high tensile strength and tensile modulus and especially geogrids have great soil interlocking capacity, which is retained even when the fill type changes from gravel to sand. It has to be noted that the horizontal stresses produced by a tracked vehicle are mainly distributed along the longitudinal and transverse axis of the tracks, hence biaxial geogrids are optimal in such situation, while uniaxial geogrids are not suitable.

For a platform constructed on soft subgrade, the geogrids stiffen the aggregate, thereby enabling the layer to distribute the crawler track loads or other loads over a larger area of the subgrade. At the same time geogrids prevent local shear failure in the subgrade soil. Hence the net effect of geogrids is to change the mode of failure in the subgrade from a local bearing failure to a general bearing failure. When the applied load is static or quasi-static (like in case of a crawler crane), then the required platform thickness can be calculated with the Static method or the T-value method, above illustrated, to prevent bearing failure.

Yet, the Static method or the T-value method assumes that the platform is stabilized with geosynthetics, but the required geosynthetics still needs to be designed based on sound engineering principles. Geogrids provide the following stabilizing mechanisms:

- base course lateral restraint mechanism for horizontal stresses generated by platform soil self-weight;
- base course lateral restraint mechanism for horizontal stresses generated by crawler tracks loading;
- membrane mechanism at the platform – subgrade interface.

Each of these three mechanisms produces tensile forces in geogrid layers.

Given the platform thickness, by considering separately the effect of the applied static loads (soil self weight and tensioned membrane mechanism) and the instant effect of quasi-static load (track loads), it is then possible to calculate the distribution of the horizontal tensile forces in the platform structure and the overall tensile forces generated in each layer of geogrids, and then to select the appropriate geogrid for each layer based on a limit state criterion. Since we are dealing with temporary working platforms, the limit state criterion cannot be the failure but rather the serviceability limit state, thus the deformations shall be limited.

In order to mobilize its tensile strength, the geogrids would need to strain and the deformation needed to mobilize this mechanism could exceed the serviceability requirements of the working platforms. However, geogrids can retain a permanent strain which is not critical to serviceability consideration; both theory and practical experience suggest that geogrid strain shall be limited to 5 %. For important or critical structures, the geogrid strain should be limited to 1 - 2 %. For less critical structures, or when the design conditions afford slightly larger deformations, geogrid strain values of 3 – 5 % can be used.

This limit strain criterion shall be applied to the short term tensile strength of the geogrid, as measured in a wide width tensile test according to EN ISO 10319 standard [10].

The tensile forces produced in the geogrids by the three active mechanisms must now be defined. A multi-layer model has been developed [4, 5] for the geogrid design (Fig. 5): asphalt course (AC), if present; base course (BC); subbase course (SB); and subgrade (SG).

The model assumes that the load is applied as a uniform vertical pressure $\sigma_{v0} = p$ on a rectangular area with half length L and half width B (L and B being the half sizes of the tracks); this load spreads in the 3 layers of the platform structure (AC, BC and SB) according to their load spreading angles $\alpha_1$, $\alpha_2$, $\alpha_3$.

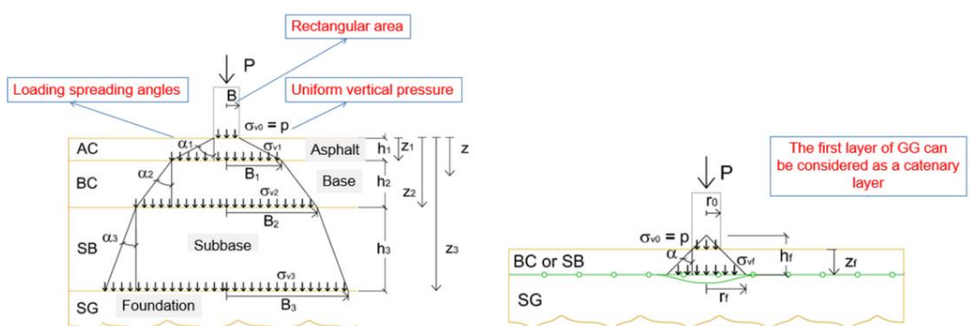


**Fig. 5.** General scheme of the 4-layers model and of the first layer of geogrid (modified from [6])

The model affords the calculation of the tensile force in each geogrid, produced by the above mentioned mechanism:

- The tensile force $T_{zi}$, generated in the i-th geogrid layer by the horizontal thrust of the soil above it, can be easily calculated based on classic geotechnical theory [6]:

$$T_{zi} = \int_{(i-1)\text{-geogrid}}^{i\text{-geogrid}} \text{horizontal soil stress} = 0.5\, K_2 \cdot [\,2\, \gamma_1\, Z_1 + \gamma_2\, (Z_i + Z_{i-1} - 2\, Z_1)] \cdot (Z_i - Z_{i-1}) \qquad (12)$$

where: $T_{zi}$ is the tensile force generated in the i-th geogrid in the base course; Z is the reference depth; $\gamma_1$ is the unit weight of the asphalt (usually not present in a working platform); $\gamma_2$ is the unit weight of base course; K is the coefficient of soil thrust.

- The force due to the horizontal stresses generated by uniform loads can be easily calculated based on classic geotechnical theory [6]:

$$T_{Pi} = \int_{(i-1)\text{-geogrid}}^{i\text{-geogrid}} \text{horiz. soil stress generated by the uniform load} = 0.5\,(\sigma_{hi} + \sigma_{hi-1})\ (Z_i - Z_{i-1}) \tag{13}$$

where: $T_{pi}$ is the tensile force generated in the i-th geogrid in the base course by the load; and $\sigma_h$ is the horizontal stress

- For the forces due to the tensioned membrane mechanism we need to consider that the first geosynthetic layer, at the interface with the subgrade, is subject to the highest vertical deformations, when the first fill layer is spread and compacted, due to the settlement of the soft subgrade; the next geosynthetic layers, instead, are far less subject to vertical displacements; hence we can reasonably assume that the first geogrid layer, which can be considered as a catenary layer, is subject to the tensioned membrane mechanism, while for the next layers such mechanism is negligible (Fig. 5). The tension in the first geogrid is determined from the following equation [11]:

$$T_m = W_{TC} \,.\, \Omega \,.\, r_f \tag{14}$$

where: $\Omega$ is the dimensionless factor, function of the strain in the geosynthetic; $W_{TC}$ is the uniform vertical load, which is a function of the load cone volume below the load with radius $r_f$ at base (Fig. 5).

The total horizontal force that the i-th geogrid layer has to withstand is then:

$$T_{tot\text{-}i} = T_{zi} + T_{Pi} + T_m \tag{15}$$

The i-th geogrid layer shall be able to provide a tensile force equal to or larger than $T_{tot\text{-}i}$ at the selected maximum strain of 1 - 5 % [4, 5, 6, 7, 8].

## 6 Case history - Working Platform, Protos Efw, UK

Significant experience has been developed with the use of Static method in the recent years. Since 2015, several temporary working platforms and access roads supporting heavy loads have been constructed using such design method. Certainly, it is worth to mention the case of Protos Efw project in UK, where the temporary platform had to be constructed over a subgrade with CBR = 0.5, and capable to resist Kobelco CCH900 cranes (450 kPa track loading). The design with the Static Method and the Geogrid design method included biaxial geogrids (> 100 kN/m tensile strength in both directions) with a thickness up to 1.10 m. As shown in Fig. 6, the test results (plate bearing test, 750 mm plate diameter with two load cycles up to 510 kPa) proved the effectiveness of this platform, able to achieve a top surface CBR > 20, with very limited deformations/settlements.

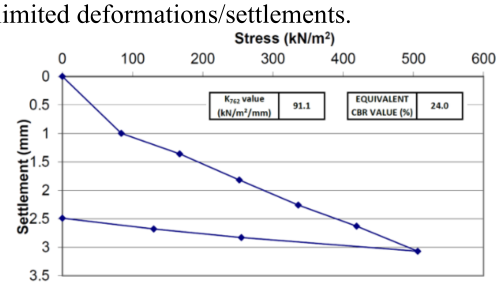


**Fig. 6.** Plate bearing test results

The excellent performance of this working platform (Fig. 7), which represents an extreme challenge in terms of high applied loads and low bearing capacity of the subgrade, demonstrates that the design methods presented in this paper are suitable and reliable.

**Fig. 7.** The crawler crane and the construction of the working platform for Protos Efw project in UK

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