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Soil Reinforcement by Full Displacement Columns below a Coal Stockpile to Reduce Impact on Adjacent Structures

Philipp Schober, Pablo Forgoso, Leinelle James Pauig
BAUER Spezialtiefbau GmbH, Germany, philipp.schober@bauer.de

Hemanth Narayanan
P.T. BAUER Pratama Indonesia, Indonesia

ABSTRACT: Soil Improvement to increase the stiffness and shear strength of the subsoil is an efficient solution for many construction projects. By replacing deep foundation systems, soil improvement can significantly reduce the construction costs and time. In the recent years, additional to the common methods like stone columns or deep soil mixing with binder, soil improvement by Rigid Inclusions has also become a well-established technique in Southeast Asia. To reduce the impact of a coal stockpile to the deep foundation of adjacent structures, improvement of the subsoil by Full Displacement Columns (FDC's) as Rigid Inclusion has been selected. Due to its efficient installation process, analysis have shown that FDC's as Rigid Inclusions were the most economical soil improvement solution to meet the project requirements. The ground improvement scheme below the stockpiles was designed using 2D the Finite Element (2D-FE) method. Next to the deformation of the natural ground caused by the weight of the coal, the additional bending moments, shear forces as well as possible negative skin friction in the spun piles next to the stockpile has been analysed. This paper presents the concept and design procedure for the soil reinforcement by FDC's as Rigid Inclusions.

KEYWORDS: Soil Reinforcement, Rigid Inclusions, Coal Stockpile, Deformation Control, Adjacent Structures.

1 RIGID INCLUSIONS AS SOIL IMPROVEMENT SOLUTION

Soil improvement can be an efficient solution for many projects to reduce foundation costs and the project duration. Especially for projects where the load is distributed over a large area, soil improvement solutions are usually much more economic than conventional deep foundation solutions (e.g., bored piles). Soil improvement is typically used at infrastructure project below embankments for roads, railways or runways. Also, the subsoil below concrete slabs of warehouses and oil/gas tanks can be improved effectively.

A relatively new soil improvement technique which was developed in Europe over the past decades and is now commonly used in most parts of the world are Rigid Inclusions. Due to its effectiveness also in very soft soils like peat, soil improvement by Rigid Inclusions is becoming recently also a well-established technique in Southeast Asia.

This technique consists of a rigid inclusion network associated with a granular earth platform (so-called Load Transfer Platform, LTP), intercalated between the reinforced soil and the upper structure. The load is transferred partially onto the inclusions through arching effect occurring in the platform due to internal strength (mainly internal friction) of the Load Transfer Platform. Generally, the Rigid Inclusions are embedded into a competent soil layer. In Figure 1, the concept of soil improvement by Rigid Inclusions for road or railway embankments (a) and below a concrete slab e.g. for a warehouse (b) is presented.

In the document "ASIRI National Project, Recommendations for the design, construction and control of rigid inclusion ground improvements design and construction principles" design and construction principles of Rigid Inclusion are described.

However, there is no special installation method or pre-defined material that must be used for the installation of rigid inclusions into the subsoil. Rigid Inclusions are simply rigid elements reinforcing the natural ground. Nevertheless, the most common and effective way is the installation of Rigid Inclusions by the so called "Full Displacement Column (FDC)" method, which will be explained in more detail in the following section.

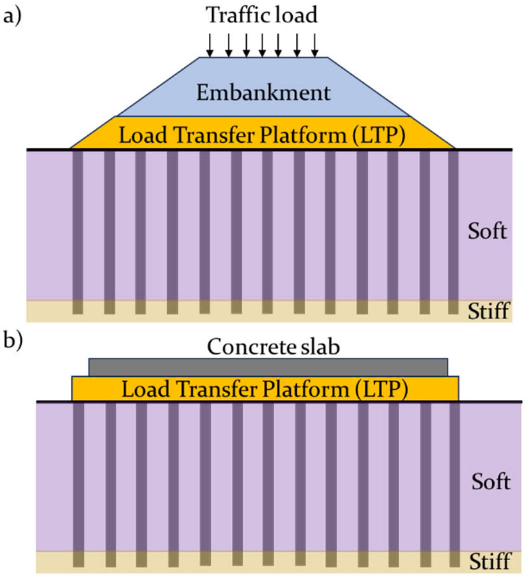

Figure 1. Examples for Soil Improvement by rigid Inclusions

2 FULL DISPLACEMENT DRILLING METHOD

The installation of rigid inclusion by the Full Displacement Column method, or short FDC, has several advantages regarding the bearing behavior of the column and the installation process on site.

The void which is filled with grout, mortar or concrete is created by the full displacement tool which is connected to the drilling rod (Figure 2). The rod itself is mounted to the rotary drive and the drilling rig.

The full displacement tool is penetrated into the subsoil with constant torque until the design depth or refusal is reached. At the final depth, the drilling tool starts slowly to be withdrawn while the mortar or concrete is pumped into the void created by the full displacement tool. During withdraw, the toll is rotated in the same direction than during penetration (Figure 3).

Figure 2. Full displacement tool on a BAUER BG28 drilling rig

Through the installation of the rigid inclusions by the FDC method, the soil is mainly not excavated but displaced. Hence, rather than a relaxation of the soil during the drilling processes the soil around the column is densified. This leads to an increase of the horizontal stress (K1 > K0) and therefore to an increase of the shaft and base resistance of the FDC compared to piles installed by the Kelly drilling method.

Moreover, with the full displacement method, no use of stabilizing fluid that could compromise the shaft friction is required. Those factors lead to an improve bearing behavior of FDC's compared to other drilling methods with soil extraction (Figure 4).

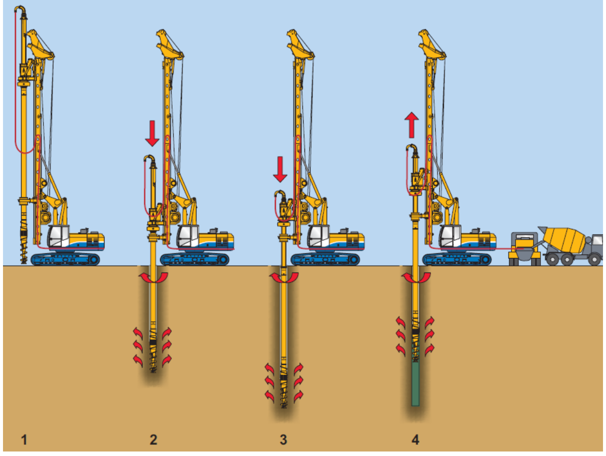

Figure 3. Sequencing for the installation of rigid inclusions by the Full Displacement Column (FDC) method

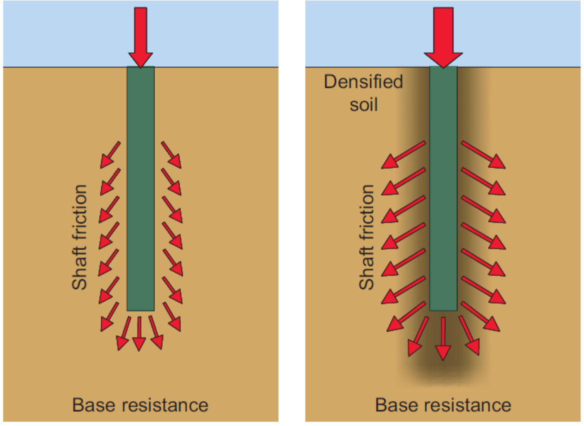

Figure 4. Effects of the displacement system on the bearing capacity

3 SOIL IMPROVEMENT BELOW A COAL STOCKPILE TO REDUCE SOIL DISPLACEMENT

3.1 *Project Description and Scope of Work*

A dome structure was planned for two adjacent coal stockpiles on the Indonesian island of Sumatra for a private coal fired power plant, each with a length of 133 m and a width of 50m. The maximal height of the stockpiles was given with 15 m. Under the top of the stockpile the maximal external load to be considered was 200 kN/m². A reclaimer is located in the middle of the two stockpiles. The schematic section through the system is shown in Figure 5.

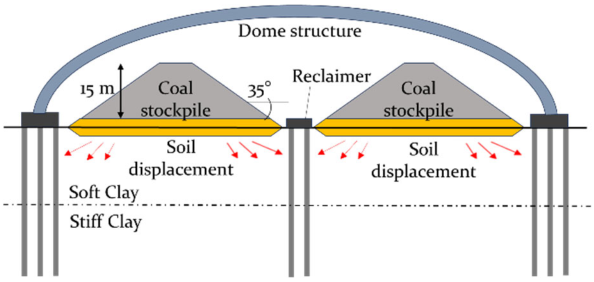

Figure 5. Schematic section showing the two coal stockpiles and the structures

The dome structure as well as the reclaimer are scheduled to be supported on spun piles with a diameter of 600 mm (D600). The length of the spun piles was provided with up to 42m arranged in rectangular grids at 2.0 m spacing. The pile foundation for the reclaimer consists of two pair of 2xD600 spun piles, 9.0m away of each other, while larger piles groups in four rows of D600 were noticed for the dome foundation.

Due to the qualified structural bending capacity of the nominated spun pile, ground improvement over the coal storage was required to limit the lateral pressure on the spun piles generated by the consolidating soil and to prevent excessive induced bending moment and shear force. Moreover, the proposed ground improvement may reduce the effect of negative skin friction on the spun piles.

3.2 *Ground conditions*

The geotechnical site investigation showed the presence of fine-grained soils in soft consistency over the upper 27 m. Those highly deformable layers are relevant for the ground improvement solution. Below the soft clay layer, more competent and therefore less deformable layers are present (stiff to hard clay).

Field and lab test results were used to interpret the engineering properties of the present soils. Shear strength parameters were estimated through well-established correlations. The relevant soil parameters for the design are listed in Table 1. The groundwater level was generally found about 3.0 m below the natural ground level.

Table 1. Soil conditions and parameter used for the Rigid Inclusions design

Soil layer		Top	qc	SPT	γ
		[m]	[MPa]	[blow]	[kN/m^3]
Firm clay	🟪	0	0.5 - 1	4	18
Soft clay	🟪	4	1 – 1.5	2	16
Stiff Clay	🟪	27	2-2.5	7	19
v. stiff clay	🟩	35	> 4	16	20
Hard clay	🟩	43	-	>20	21

Soil layer		γ'	φ'_k	c'_k	$c'_{u,k}$
		[kN/m^3]	[MPa]	[blow]	[kN/m^3]
Firm clay	🟪	8	25	3	25
Soft clay	🟪	16	25	5	40
Stiff Clay	🟪	19	25	7	80
v. stiff clay	🟩	10	27	12	90
Hard clay	🟩	11	30	25	250

3.3 *Soil Improvement Solution using Full Displacement Columns as Rigid Inclusions*

After studying the project specification and soil condition as well as considering operational factors, the improvement of the subsoil below the stockpiles by rigid inclusions was evaluated as most efficient improvement concept. Hence, ground improvement scheme choice for this project follows the principles of the rigid inclusion theory and consists of rigid elements executed as FDC and a Load Transfer Platform (LTP) over the inclusions as intermediate element that may distribute external loads into the natural soil and the FDC based on the proportionate stiffness.

The LTP is composed of very dense well-graded granular soil material (Relative density of Dr > 85%) with low fine content (<10%). The thickness of the LTP was optimized to 1.50m to guarantee a safe bearing capacity at FDC head. The LTP was embedded 0.75 m below grade and was reinforced with two layers of geogrids with ultimate tensile strength of 500kN/m in both directions.

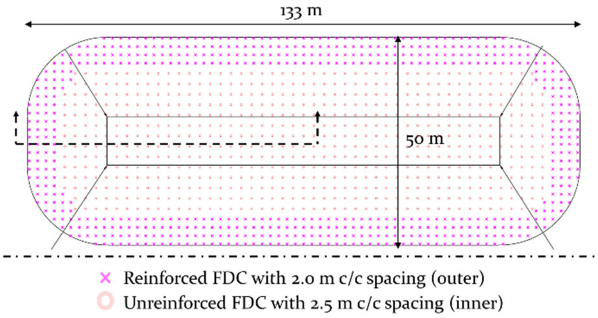

Figure 6. Layout of FDC's below one coal stockpile

The FDC's are drilled with a diameter of 420mm from ground level down to 27 m depth (Figure 8). The top of the FDC for design was though assumed 75 cm below grade matching the LTP base level. The unreinforced columns were arranged in a rectangular grid of 2.50 m spacing. To accommodate the high bending moments with low compressive axial force, the outmost 4 rows of FDC's are arrayed in 2.00m spacing in both directions and carry a partial reinforcement cage of 12 m length (Figure 6 and Figure 7).

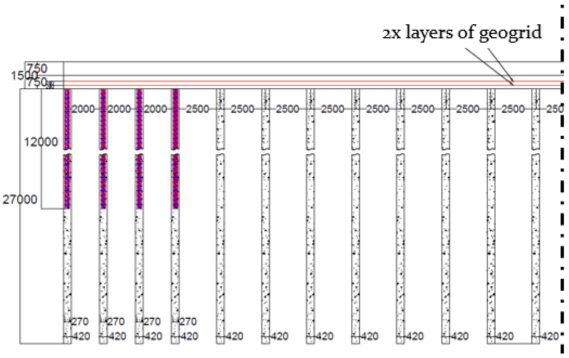

Figure 7. Section through the half of the coal stockpile showing the LTP, FDC's and reinforcement cage

Figure 8. Installation of the FDC columns with a maximal length of 27 m using a BG28 with Kelly extension

3.4 *External Loads*

The uniform weight of the coal storage was provided as 200 kPa, decreasing linearly at an angle of 38° degrees over 19.2 m length. The total width of the coal storage is 50m.

The structural loads on the spun pile foundations were omitted in the calculation model to clearly identify the magnitude of induced internal forces by the consolidating soil.

3.5 *Desing of FDC's as Rigid Inclusions using 2D fine element software*

A comprehensive numerical study was performed by means of 2D-FE modelling with Plaxis 2D to capture and evaluate the soil-structure interaction in more realistic manner. The FE-model size was established to embrace the influence depth and extension of the proposed structures. Moreover, only one half of the shed was considered to evaluate the effects on spun piles by unsymmetric loading. The meshing coarseness was refined in the clusters around the FDC and spun piles.

2D-FE modelling provides satisfying results to determine settlement reduction and stress-transfer mechanisms for rigid inclusion systems. However, it also imposes several important limitations compared to a 3D-FE analysis that have to be considered when analyzing and applying the output of 2D-FE calculations in rigid inclusions design.

In plane-strain conditions, each inclusion is represented as a continuous wall extending infinitely out of the plane, which artificially increases lateral confinement and stiffness compared to discrete 3D columns (Briançon & Simon, 2012; Chen & Martin, 2002). This simplification can lead to overestimation of stress concentration ratios (SCR) and inclusion load share (ASIRI, 2012). Similarly, 2D modelling cannot accurately represent the three-dimensional geometry of inclusion grids, where diagonal load paths and spatial arching significantly influence settlement profiles and load transfer to the LTP (Han & Ye, 2002; Van Impe & Van Impe, 2019).

Another limitation is the inability of 2D analyses to model group effects and variable tributary areas. In practice, inclusions interact through overlapping stress bulbs and spatial load redistribution, particularly under non-symmetric loading conditions or variable spacing; such effects are inherently 3D and cannot be simulated adequately with a single 2D section (Kirsch & Kirsch, 2010; Kempfert & Gebreselassie, 2006). Installation-induced soil displacement and densification also exhibit strong 3D behavior, and 2D modelling typically does not capture lateral stress changes around individual inclusions (Huang & Han, 2010; Plaxis BV, 2023).

Because of the permanent nature of the ground improvement works and the vulnerability of the imposed loading to a precise time schedule, the numerical analysis was carried out under full drained conditions. Therefore, a consolidation analysis was not pursuit and the development of settlement vs. time was not computed.

Soil materials were modelled by the Hardening Soil (HS) constitutive law applying the strength parameters in Table 1, except for the soft clay as Soft Soil (SS). The stiffness parameters were introduced in terms of Cc (Compression Index), Cs (Swelling Index) & e_0 (initial void ratio) for the soft clay as estimation upon soil properties in Table 1. For stiffer layers, $E_{50,ref}$ (elastic modulus at 50% of the failure load) was estimated for a reference vertical stress of σ'=100 kPa based on the laboratory test result and SPT values. Moreover, the ratio of $E_{ur,ref}$ (unload-reload modulus)/ $E_{50,ref}$ was assumed to be between 3-4.

FDC's were idealized as embedded beam elements attached through a hinge to a dummy plate on top to accurately capture the external load distribution through the LTP. Interface elements were activated around all plate elements in contact to the ground.

A young's modulus of E=20 GPa was adopted as FDC material stiffness with elastic-plastic properties to limit their contribution during the safety calculation to a compatible level.

The spun piles were modelled as embedded beam elements likewise. At the top, spun piles were rigidly connected to the pile cap, which was simulated as stiff plate element. A Youngs modulus E=35 GPa was adopted for the spun piles as material stiffness with elastic-plastic properties to avoid any significant contribution during the safety calculation.

The geogrids in the LTP were introduced as geogrid elements discretized into two layers with 20 cm offset at 20cm distance from the FDC's. A long-term axial stiffness of 6000 kN/m was estimated for each layer. A plastic tensile capacity was assigned to the geogrids to allow for a realistic contribution during the safety calculation.

Following construction steps were considered in the finite element model:

- Phase 1: Initial conditions (Ko-procedure)
- Phase 2: Installation of spun piles (reclaimer and dome)
- Phase 3: Installation of FDC's
- Phase 4: Construction of LTP
- Phase 5: Apply vertical component of coal stockpile (horizontal component after friction deduction at LTP base directly assigned to the geogrid)
- Phase 6: Safety analysis (φ-c reduction analysis)

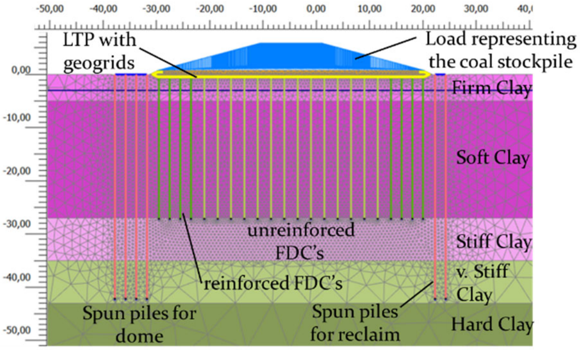

Figure 9. 2D Finite Element used for the design of the FDC below the coal stockpile

4 RESULTS OF THE DESIGN CALCULATIONS

4.1 *Geotechnical Design Results*

The maximum value of relevant figures computed for the different elements are summarized in Table 3.

The positive impact of the FDC's on the spun piles can be recognized by the induced deformations, shear force and bending moment to moderate magnitude. The calculation also shows the loss of skin friction as resistance member over 32 m depth which had to be simultaneously considered in the spun pile foundation design.

Table 2. Summary of design results

		Overall	FDC	
			Unreinf.	Reinf.
Settlement	[mm]	293	-	33
Lateral deflection	[mm]	36	-	36
Max. axial force	[kN/m]	-	482	510
Max. bending moment	[kN/m/m]	-	15	31
Max. shear force	[kN/m]	-	6	10
Safety factor	[-]	1.75	-	-

		Overall	**Spun piles**	
			Reclaimer	Shed
Settlement	[mm]	293	35	33
Lateral deflection	[mm]	36	34	36
Max. axial force	[kN/m]	-	535	510
Max. bending moment	[kN/m/m]	-	37	31
Max. shear force	[kN/m]	-	10	10
Safety factor	[-]	1.75	-	-

The positive impact of the FDC's on the spun piles can be recognized by the induced deformations, shear force and bending moment to moderate magnitude. The calculation also shows the loss of skin friction as resistance member over 32 m depth which had to be simultaneously considered in the spun pile foundation design.

The vertical deformation of the LTP and the subsoil below the coal stockpile are presented in Figure 9. In the middle of the stockpile where the highest load of 200 kN/m² applies, the vertical settlement is in a range of about 300 mm. The estimated settlement settlements around the spun piles are < 50 mm. Those settlements around the spun piles lead to the negative skin friction which had to be considered in the spun pile design.

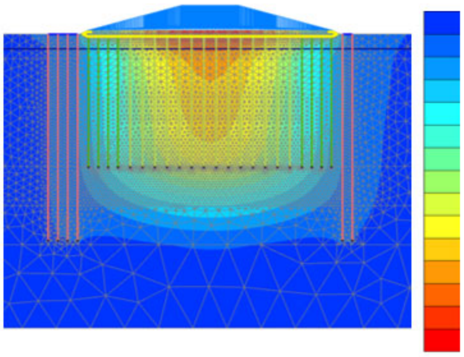

Figure 10. Vertical displacements due to the load of the coal stockpile in the improved subsoil

The horizontal displacement caused by the loading are presented in Figure 10. The displacement in a maximal range of about ± 36 mm is reflected in the additional shear forces and bending moments that have to be considered in the spun pile design.

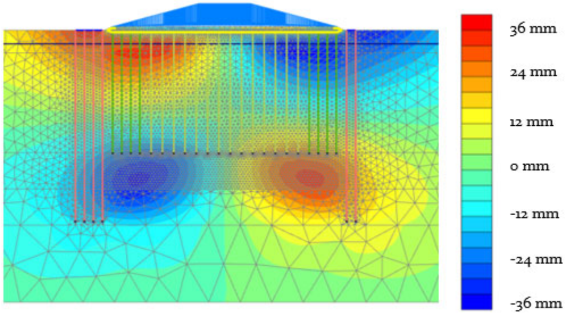

Figure 11. Horizontal displacements due to the load of the coal stockpile in the improved subsoil

The axial force distribution, and hence, the location of the maximum load is to be considered if combined with bending moments for later concrete design.

The maximal principle stress in the LTP is around 800 kN/m^3 directly above the head of the FDC column as shown in Figure 12.

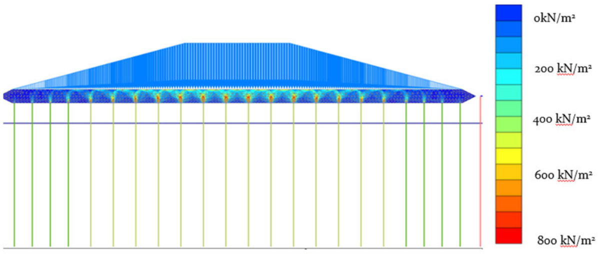

Figure 12. Principle stresses in the Load Transfer Platform (LTP)

The overall stability of the system checked by the φ-c reduction procedure in calculation phase 6 reflects the available safety at the most critical point in the numerical model. The critical failure mechanism computed in phase 6 represents a local failure of the slope of the outcropping LTP. Any other part of the model will yield a higher safety factor, considered as acceptable and sufficient for the project characteristics.

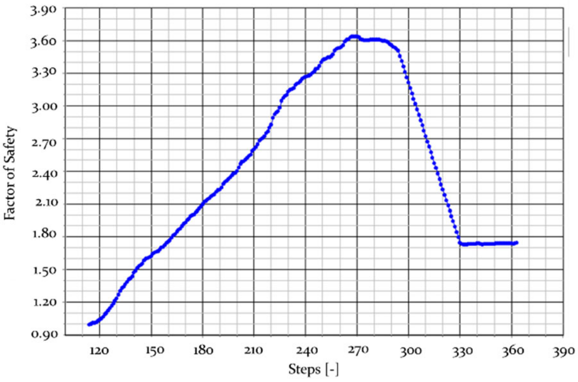

Figure 13. Factor of Safety (FoS) as result of the overall stability calculation

4.2 *Structural Design Results*

Structural design of the rigid inclusions was conducted according to DIN EN 1992-1-1 based on following materials:

- Concrete grade: C30/37, f_{ck} = 30MPa after 28 days
- Steel reinforcement: f_{yk} = 420MPa

For the reinforced columns, an environmental class of XC2, XD1, XA1 was assumed. A nominal concrete cover of 75mm to the spiral was selected for the cages. A minimum cage of 6 x D29mm + Spiral of D10/250mm was proposed for enough robustness during installation. Due to the bending moment depth observed in the design calculation, a 12m deep cage is required in the outer 4 rows of each coal stockpile.

The internal forces computed in the finite element analysis were multiplied by the correspondent column spacing and later factored by the proper load safety coefficient γ_g. The compressive axial force was factored by $\gamma_{g,inf}$ due to its beneficial effect in the bending capacity. Partial safety factors applied in the structural design are listed in Table 3.

Table 3. Partial safety factors applied in the structural design

Layout		Concrete			Reinforcement
γ_g	$\gamma_{g,inf}$	$\alpha_{cc,plain}$	$\alpha_{cc,rfo}$	γ_C	γ_S
1.35	1.00	0.70	0.85	1.50	1.15

4.3 *Geogrid Design Results*

The operational long-term ultimate resistance of a geogrid was calculated to the Recommendations for Design and Analysis of Earth Structures using Geosynthetic Reinforcements (EBGEO) as follows:

$$R_k = \frac{R_{k,0}}{A_1 \cdot A_2 \cdot A_3 \cdot A_4 \cdot A_5} \qquad (1)$$

Where: $R_{k,0}$ is with 500 kN/m the chosen strength of the geogrid based on the design calculations (Figure 14) and A1 is the reduction factor for creep effect (A1 = 1.50), A2 is the reduction factor for possible damage (A2 = 1.50), A3 is the reduction factor for manufacture (A3 = 1.00), A4 is the reduction factor for environmental effects (A4 = 1.50) and A5 is the reduction factor for dynamic effects (A5 = 1.00). With above input the characteristic strength of the geogrid is R_k = 148kN/m.

Dismissing local peaks, the maximum load in one geogrid layer may be rounded to Tk = 70kN/m. The design load is calculated as Td = γ_g ·Tk = 1.35 70 kN/m = 95 kN/m. The design resistance of the geogrid is calculated as Rd = Rk / γ_M =

148kN/m / 1.40 = 106 kN/m. R_d is therefore > than T_d (106 kN/m > 96 kN/m).

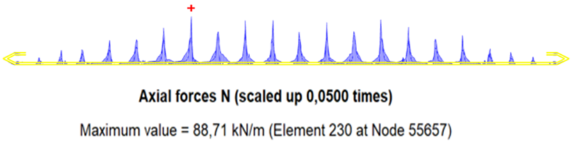

Figure 14. Axial force N in the geogrid layer of the Load Transfer Platform (LTP)

The proposed 2x geogrid layers ft. $R_{k,0}$ = 500kN/m ultimate strength in both directions fulfil the design provisions following the design procedure according to EBGEO.

5 CONCLUSION

Full Displacement Columns as Rigid Inclusions with a Load Transfer Platform were designed to improve the natural ground under the two coal stockpiles to protect future adjacent foundations on spun piles from excessive lateral pressure induced by the consolidating soil.

The positive impact of the FDC's on the spun piles can be observed by the moderate induced deformations, shear force and bending moments. However, it is to highlight the significant effect of negative skin friction remaining in the spun piles. The computed compressive axial force, but also the loss of skin friction as resistance member over 32m depth had to be simultaneously considered in the spun pile foundation design.

An effective soil improvement scheme by FDC's as Rigid Inclusion solution was implemented to meet the project requirements in terms of technical, commercial and operational aspects.

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