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GEOGRID-REINFORCED SOIL MAT FOR TEMPORARY SUPPORT OF HEAVY EQUIPMENT

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ABSTRACT

Concrete blocks were used in a temporary storage facility to support replacement steam generators for a nuclear power plant. From the subsurface investigation, it was found that direct ground support of these concrete blocks was not feasible. In order to avoid the use of piles and associated concrete pile cap, which would be costly and take longer to build, a geogrid-reinforced soil mat foundation was adopted. The concrete blocks were put on a steel plate, which in turn rested on the geogrid-reinforced soil mat. Upon completion of the geogrid-reinforced structural fill mat, the replacement steam generators were moved into the temporary storage facility. Settlement monitoring of the concrete blocks proceeded for more than five months and disclosed settlements that were larger than those calculated. However, the measured settlements were still relatively small and the geogrid-reinforced soil mat foundation performed satisfactorily.

INTRODUCTION

Replacement of steam generators at nuclear power plants requires on-site storage of these heavy pieces of equipment. Final equipment preparation is done on-site during the storage period, and requires that the equipment be supported above ground to allow full access during final preparation.

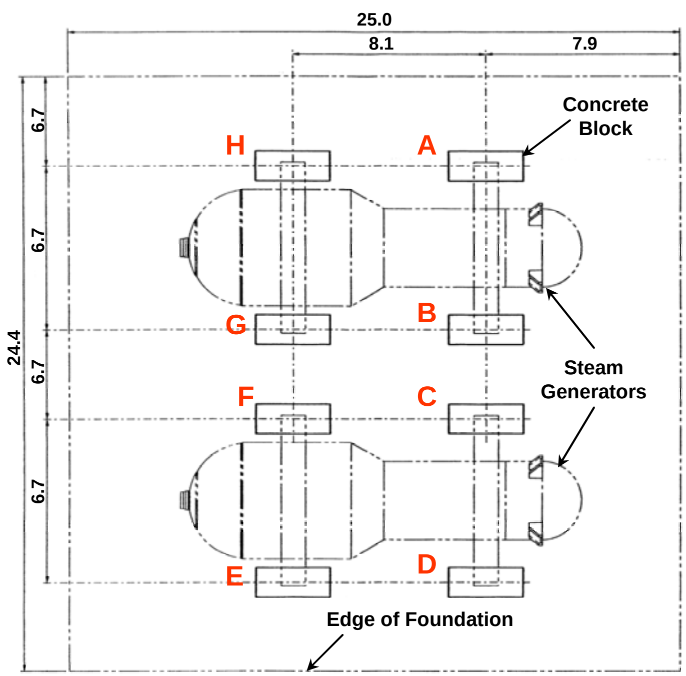

Fig. 1. Plan view of the concrete blocks supporting the steam generators (unit: m)

Equipment support usually consists of prefabricated concrete blocks that are preferably placed on the ground with minimum subgrade preparation.

For a recent steam generator replacement project, the layout of the replacement steam generator storage facility (RSGSF) is shown in Fig. 1. With identical plan dimensions of about 1.2 m x 3 m, eight rectangular concrete blocks denoted as A to H were distributed within this facility to support the 2 steam generators. The design bearing pressure for the concrete blocks was about 290 kPa. Figure 2 shows the storage facility with one of the steam generators in place.

Fig 2. Storage facility with one steam generator in place

SITE CONDITIONS

The subsurface conditions at the on-site storage location were investigated with two SPT borings, (i.e., B-1 and B-2). The generalized subsurface profile consisted of two soil layers (Layer I and Layer II) and bedrock, as shown in Fig. 3. Layer I consisted of predominantly soft to medium stiff clays and/or very loose to medium dense silts with interbedded silty sand to medium sand seams (possible fill). Layer I extended to a depth of 8.3 m below grade at B-1, and 3.5 m below grade at B-2. Layer II consisted of very loose to medium dense sands with interbedded clay seams. Layer II reached the top of bedrock at a depth of 19.8 m below grade in boring B-1, and extended to the termination depth of 9.1 m at boring B-2. Ground water was encountered at depths greater than 2.3 m below grade. The subsurface profile suggested that low allowable bearing pressures should be anticipated.

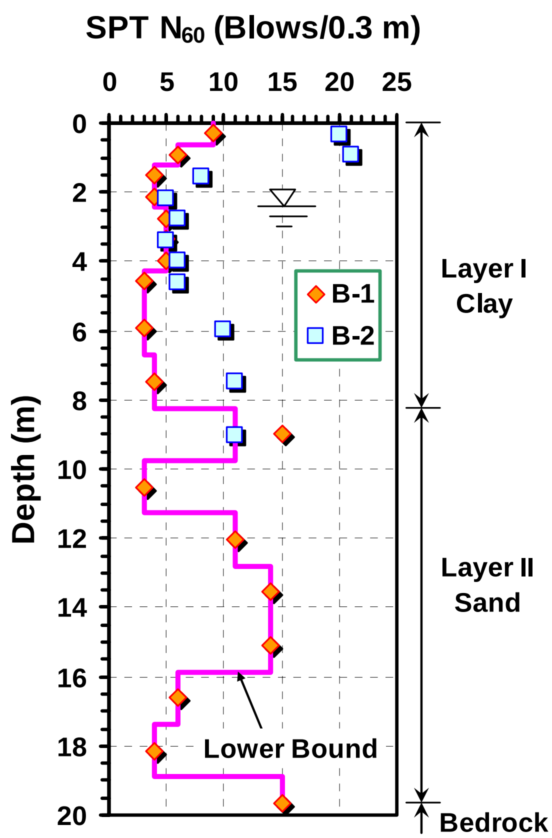

Fig. 3. SPT blow counts N_{60} from the subsurface investigation

ENGINEERING PROPERTIES

Engineering properties of the subsurface soils were assessed from the SPT boring logs. Boring B-1 was terminated at a greater depth than boring B-2, and the N-values from B-1 are consistently lower than those from B-2.

The standard energy efficiency for SPT tests is 60%, and many correlations of engineering properties are based on the blow counts corrected to 60% energy efficiency (N_{60}). Automatic hammer used to perform the SPT tests at this site had energy efficiency of around 80%. Therefore N_{60} = (80%/60%)N = 1.33N. Using N_{60} = 1.25N herein, the derived N_{60} is presented in Fig. 3 for both borings. The median value of the N_{60} is 5 and 11 blows/0.3 m for Layer I and II, respectively.

No undisturbed sampling was performed in Layer I, and the undrained shear strength, s_u, of the fine-grained soil in Layer I was determined using the empirical correlation (Lowe & Zaccheo, 1975):

$$s_u = 6N_{60} \text{ (kPa)} \tag{1}$$

Since N_{60} = 5 blows/0.3 m, s_u = 6 x 5 = 30 kPa.

The angle of shearing resistance, ϕ', of the granular soils in Layer II was estimated from empirical correlations with SPT N_{60} values (Bowles, 1982). For SPT N_{60} = 11 blows/0.3 m, the value of ϕ' is in the range from 30° to 35°. A value of ϕ' = 30° was used for design.

The following correlation was used to estimate the elastic modulus of clayey soils in Layer I (Davie & Lewis, 1988):

$$E = 600 s_u \tag{2}$$

Since s_u = 30 kPa, E = 600 × 30 = 1,800 kPa = 1.8 MPa.

The elastic modulus of the sandy soil in Layer II was estimated using the empirical correlations with SPT N-value (Davie & Lewis, 1988):

$$E = 1.722 N_{60} \text{ (MPa)} \tag{3}$$

Since N_{60} = 11 blows/0.3 m, E = 1.722 × 11 = 18.9 MPa. A value of E = 19 MPa was used for design.

Table 1 summarizes the engineering design parameters that are derived from the SPT N-values.

Table 1. Design Values for the Soil Layers

Soil Layer	Thickness (m)	SPT N_{60} (blows/ 0.3 m)	s_u (kPa)	ϕ' (°)	E (MPa)
Layer I	8.2	5	30	0	1.8
Layer II	11.6	11	0	30	19

DESIGN OF GEOGRID REINFORCED FOUNDATION

Given the poor subsurface conditions as investigated, it became clear that direct ground support of these blocks was not feasible. A geogrid-reinforced soil mat foundation was then considered for this site, since it is a cost-effective alternative to piling or other ground improvement techniques.

In geogrid-reinforced soil mat foundations, a number of layers of geogrids are placed below a footing to create a composite material with improved performance characteristics (Wayne 1998). Dimensions used in geogrid-reinforced soil mat foundation design are illustrated in Fig. 4. The typical and recommended design values of these dimensions are listed in Table 2.

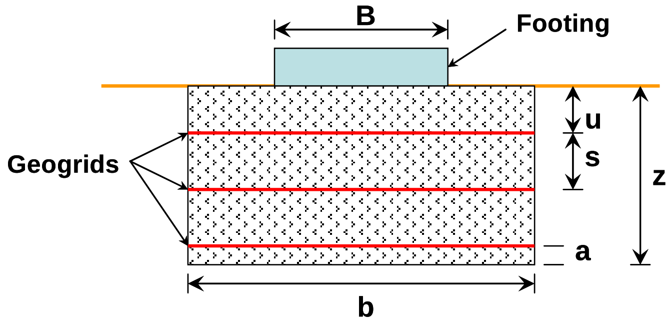

Fig. 4. Dimensions of geogrid reinforced foundation

The following two concrete block support options were then considered: 1) The blocks are put on top of a geogrid-reinforced structural fill mat (Fig. 5); 2) The blocks rest on a steel plate, which in turn rests on a geogrid-reinforced structural fill mat (Fig. 6). The second option was selected based on bearing capacity and settlement calculations.

Table 2. Typical Design Parameters for Punched and Drawn Biaxial Geogrids (Wayne et al., 1998)

Item	Typical Value	Recommended (not greater than)
u	0.15B to 0.3B	0.5B
s	0.15B to 0.3B	0.5B
z	0.5B to 1.0B	2.0B
b	2.0B to 3.0B	4.0B
a	0.1B to 0.2B	0.3B
ΔL	0.5B to 1.0B	2.0B
N	2 to 4	5

Note: B – footing width; u – distance from footing base to uppermost geogrid; s – spacing between geogrid layers; z – thickness of reinforced fill; b – width of reinforced fill; a – distance from lowest geogrid to bottom of reinforced fill; ΔL – length of geogrid extended beyond each end of footing; N – number of geogrid layers.

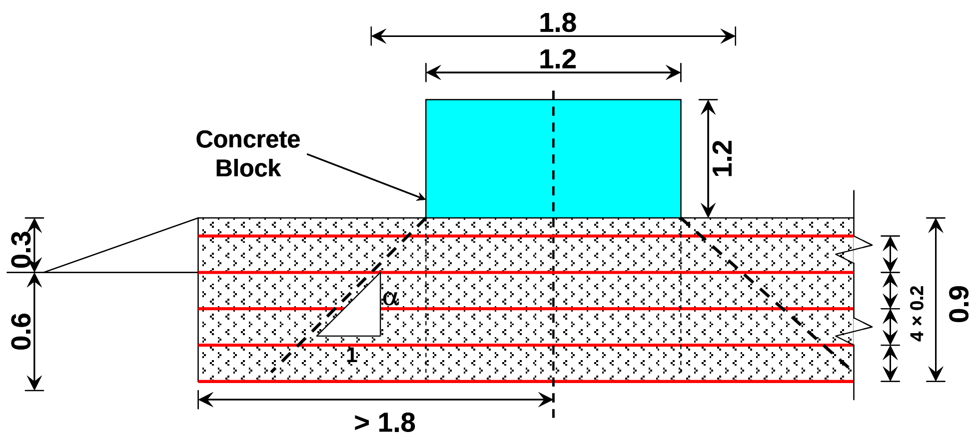

Fig. 5. Schematic of discarded foundation design (unit: m)

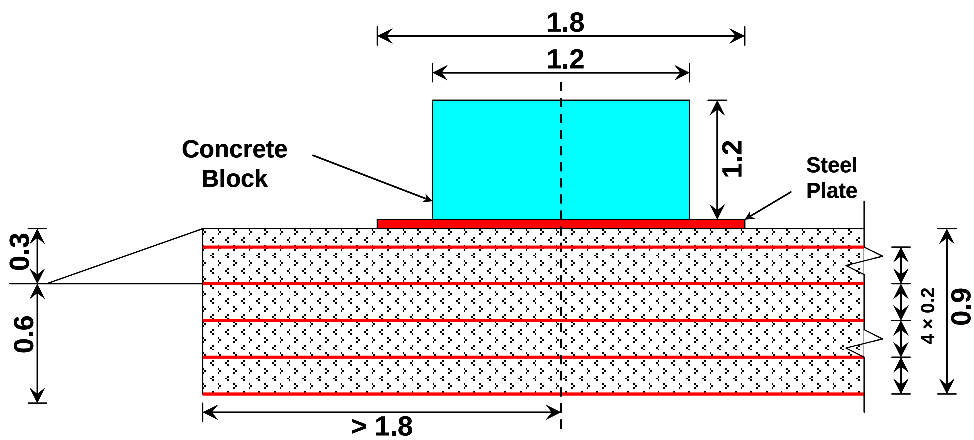

Fig. 6. Schematic of adopted foundation design (unit: m)

Site grading requirements, required that the final surface of the geogrid-reinforced soil mat be at an elevation 0.3 m higher than the original ground surface. Based on geogrid-reinforced soil mat design requirements, and bearing capacity and settlement considerations to be discussed later, it was necessary to excavate the existing soils to a depth of 0.6 m, and then backfill with geogrid-reinforced granular material for 0.9 m. A steel plate with the size of 1.8 m x 3.7 m was also required under the footing, for the purpose of distributing load, which will also be discussed later. Thus the value of B for all analyses was taken as the width of the steel plate (1.8 m), and the values listed in Table 3 were used as guidelines for design of the geogrid-reinforced soil mat foundation at this site. The final configuration of the geogrid-reinforced soil mat is presented in Fig. 6.

Table 3. Selected Values for Geogrid Reinforced Foundation at the Site

Item	Recommended value with B = 1.8 m	Used
u (m)	< 0.9	0.1
s (m)	< 0.9	0.2
z (m)	< 3.7	0.9
b (m)	< 7.3	> 3.7
a (m)	< 0.5	0
ΔL (m)	< 3.7	> 0.9
N	< 5	5

BEARING CAPACITY OF RSGSF FOUNDATION

Bearing capacity of shallow foundations can be calculated according to the following equation (Vesic, 1975):

$$q_o = cN_c\zeta_c + qN_q\zeta_q + 0.5\gamma BN_\gamma\zeta_\gamma \qquad (4)$$

where: q_o = ultimate bearing pressure
c = soil cohesion below the foundation
q = effective overburden pressure at the bottom of foundation level
γ = unit weight of soil
B = foundation width
N_c, N_q, N_γ = bearing capacity factors
ζ_c, ζ_q, ζ_γ = foundation shape

For a rectangular foundation,

$$\zeta_c = 1+(B/L)(N_q/N_c)$$
$$\zeta_q = 1+(B/L)\tan\phi$$
$$\zeta_\gamma = 1-0.4B/L$$

Values of N_c, N_q, N_γ are shown on Table 4.

The allowable bearing capacity (q_a), not considering settlement, is given by

$$q_a = q_o/3 \qquad (5)$$

Table 4. Bearing Capacity Factors

ϕ	N_c	N_q	N_γ
0^o	5.14	1.00	0.00
30^o	30.14	18.40	22.40

Bearing capacity calculations were made assuming a fully cohesive subsurface profile (Layer I extends all the way down to the bedrock), then a fully cohesionless subsurface profile (Layer II extends all the way up to the ground surface). This was expected to provide lower bound (Layer I extending all the way down to the bedrock) and upper bound (Layer II extends all the way up to the ground surface) values for the bearing capacity.

For case 1) shown in Fig. 5, the concrete block is 1.2 m x 3 m in plan dimensions (B_f=1.2 m, L_f=3 m), and it is put on top of the geogrid-reinforced soil mat foundation. Experiments have shown (Wayne et al., 1998) that a stress distribution angle of about 45^o can be achieved with geogrids (i.e. α = 1 with the slope of the stress distribution line defined as α:1, where α is shown in Figure 5). Therefore, the equivalent footing at the bottom of the geogrid reinforced foundation has the following plan dimensions:

$$B = B_f + 2z/\alpha = 1.2 + 2\times0.9/1 = 3 \text{ m}$$
$$L = L_f + 2z/\alpha = 3 + 2\times0.9/1 = 4.8 \text{ m}$$

Therefore, the pressure applied to the bottom of the geogrid reinforced foundation is the sum of the pressure resulting from the load applied to the footing and the extra overburden stress $\Delta\sigma_{vo}$' caused by the backfill:

$$q = q_f B_f L_f/(BL) + \Delta\sigma_{vo}'$$

where:
q = bearing pressure at the bottom of the geogrid-reinforced soil mat
q_f = pressure applied by the concrete block on the surface of the geogrid-reinforced soil mat = 290 kPa
$\Delta\sigma_{vo}$' = pressure increase from construction of the geo-reinforced soil mat (assuming that 0.6 m of excavated soils had the same unit weight as the backfill soil, and the unit weight of the backfill is 19 kN/m^3)

$$q = 290\times1.2\times3/(3\times4.8) + 19\times0.3 = 78 \text{ kPa} \qquad (4)$$

The equivalent footing is placed at a depth of D_f = 0.6 m and ground water table is 2.3 m in depth. The allowable bearing pressure at the bottom of the geogrid reinforced foundation is thus calculated, assuming Layer I extends all the way down to the bedrock. The allowable bearing pressure (q_a) is 59 kPa, which is equal to the design pressure at the bottom of the geogrid reinforced foundation (q = 78 kPa). Therefore, this design is discarded.

For case 2) shown in Fig. 6, the concrete block rests on a steel plate of the size of 1.8 m x 3.7 m (B_p=1.8 m, L_p=3.7 m), which in turn rests on the geogrid reinforced foundation. Similar to case 1), a stress distribution angle of 45^o is taken in the geogrid reinforced foundation (α =1). Therefore, the equivalent footing at the bottom of the geogrid reinforced foundation has the following plan dimensions:

$$B = B_p + 2z/\alpha = 1.8 + 2\times0.8/1 = 3.7 \text{ m}$$
$$L = L_p + 2z//\alpha = 3.7 + 2\times0.9/1 = 5.5 \text{ m}$$

Therefore, the pressure applied to the bottom of the geogrid reinforced foundation is the sum of the pressure resulting from the load applied to the footing and the extra overburden stress $\Delta\sigma_{vo}$' caused by the backfill:

$$q = 290\times1.2\times3/(3.7\times5.5) + 19\times0.3= 57 \text{ kPa}$$

The allowable bearing pressure at the bottom of the geogrid-reinforced soil mat is thus calculated, assuming Layer I extends all the way down to the bedrock. The allowable bearing capacity (q_a) is 59 kPa, which is greater than the pressure applied to the bottom of the geogrid reinforced foundation (q = 57 kPa).

Taking the subsurface profile to be cohesionless (c = 0.0 kPa, ϕ = 30^o), the allowable bearing pressure calculated for this foundation is q_a = 230 kPa, which is the upper bound of the allowable load. It is noted that even if we assumed the profile to be granular from top to bottom, the allowable bearing pressure (q_a = 230 kPa) would still be smaller than the pressure applied by the concrete blocks (q = 290 kPa).

SETTLEMENT ANALYSIS OF RSGSF FOUNDATION

The settlement of foundations at the site was expected to be mostly elastic and short-term. The Layer I soils contained clays, but the ground water level was usually such that consolidation settlements were expected to be minimal.

The settlement, ΔH, due to "elastic" compression of the subsurface materials was calculated from:

$$\Delta H = \Sigma(\Delta p\times H/E)$$

where, ΔH = calculated settlement (mm)
Δp = pressure increase at the center of the layer due to the loads applied at the bottom of the foundation (kPa)
H = layer thickness (mm)
E = soil deformation modulus for the soils (kPa)
The bedrock was considered incompressible.

The pressure increase at depth, Δp, was calculated in accordance with Poulos & Davis (1974) for flexible foundations. A spreadsheet was developed to calculate Δp, and the settlement at the center and along the longer edge of rectangular foundations. The settlement of the actual rectangular foundation was taken as the average between the two calculated settlements. Each soil layer was divided into smaller thickness layers to increase the precision of the settlement calculations. All settlement calculations took into consideration the existing soils to a depth below the bottom of foundation which is the larger of

- twice the foundation width;
- a depth at which the vertical stress increase from the foundation loading is no more than 10% of the foundation loading; or;
- a depth at which the vertical stress increase from the foundation loading is no more than 10% of the effective overburden pressure.

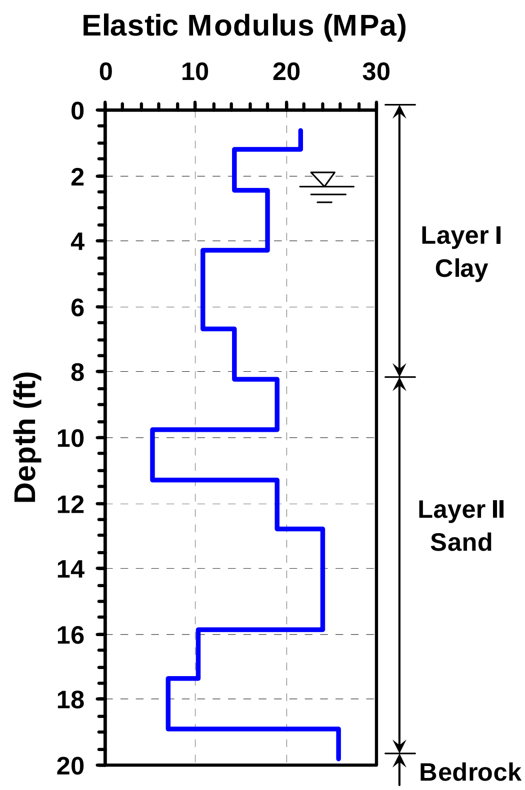

Fig. 7. Elastic modulus derived from lower bound SPT N-values

The equivalent footing at the bottom of the geogrid reinforced foundation has the plan dimensions of 3.7 m x 5.5 m. The pressure applied to the bottom of the geogrid reinforced foundation is about 57 kPa based on the calculation in the previous section. Using these design loads and the E-values shown on Table 1, the resulting calculated settlement is about 13 mm.

The settlement was also calculated using E-values based on lower bound SPT N-values along the depth, rather than the E-values shown on Table 1, which are based on median SPT N-values. The lower bound SPT N-values along the depth are shown in Fig. 3, and the corresponding values of elastic modulus E (Fig. 7) are then calculated according to equations (2) and (3). Under the design load of 57 kPa at the bottom of the geogrid-reinforced soil mat, the resulting settlement is about 15 mm.

SETTLEMENT MONITORING

Upon completion of the geogrid-reinforced structural fill mat, the steam generators were moved into the temporary storage facility and were supported by the concrete blocks. Settlement monitoring of the concrete blocks proceeded for more than five months.

Settlements were measured at each of the corners of the concrete blocks, which are numbered as 1 to 4 as shown in Fig. 8.

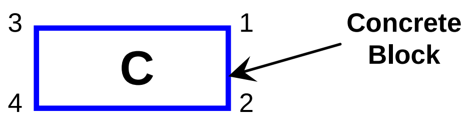

Fig. 8. Numbered corners of concrete block for settlement monitoring

The measured settlements for each block are plotted in Fig. 9. It was noted that the equipment support saddles placed on top of the blocks (refer to Fig. 2) did not extend all the way to the outside edges of the blocks, and thus the inside part of the blocks were more heavily loaded. This resulted in a slight tilt of the blocks and the inside points of any given block generally settled more than the outside points. The results shown on Fig. 9 also show essentially stabilized settlements after 5 months from load application.

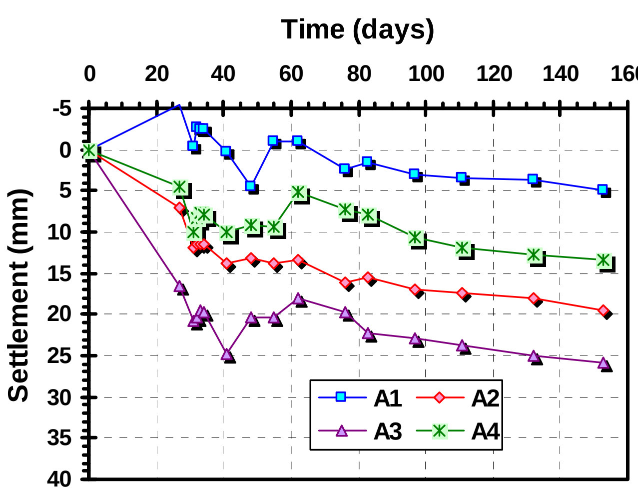

Fig. 9(a) – Settlement of block A

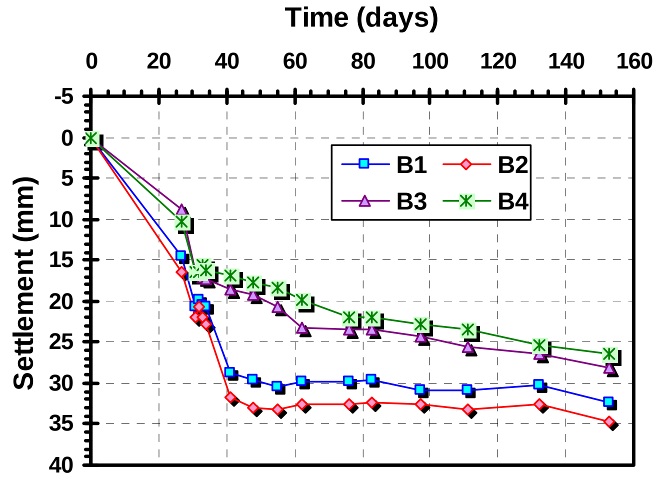

Fig. 9(b) – Settlement of block B

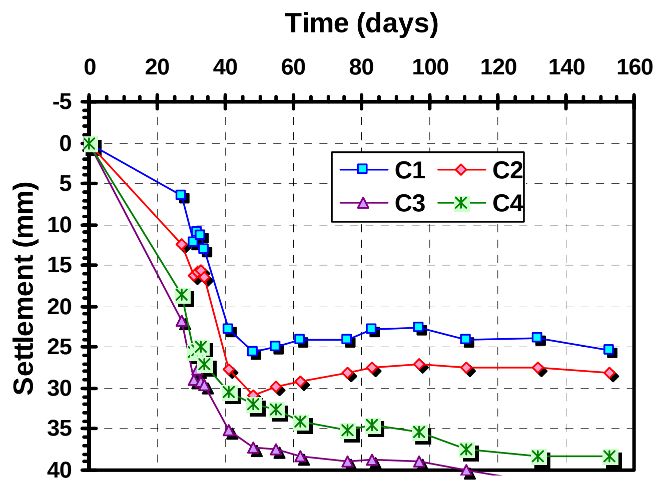

Fig. 9(c) – Settlement of block C

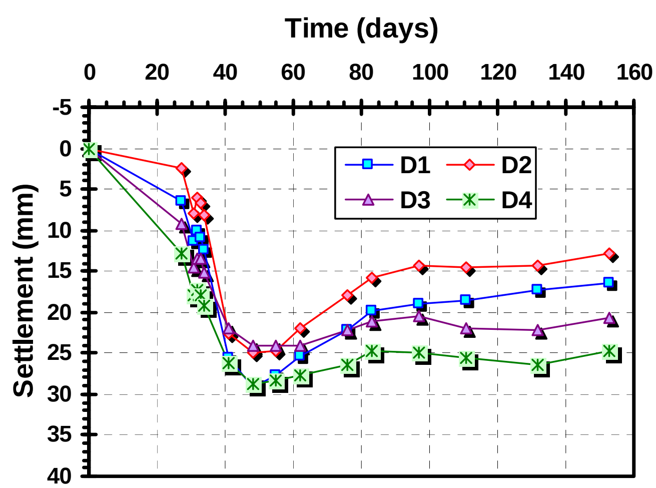

Fig. 9(d) – Settlement of block D

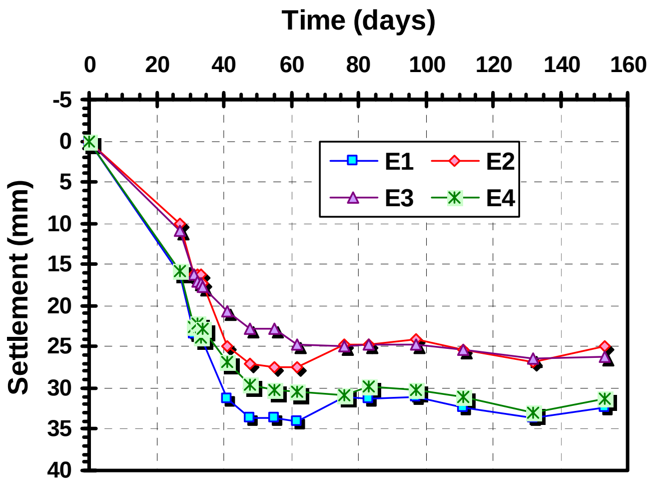

Fig. 9(e) – Settlement of block E

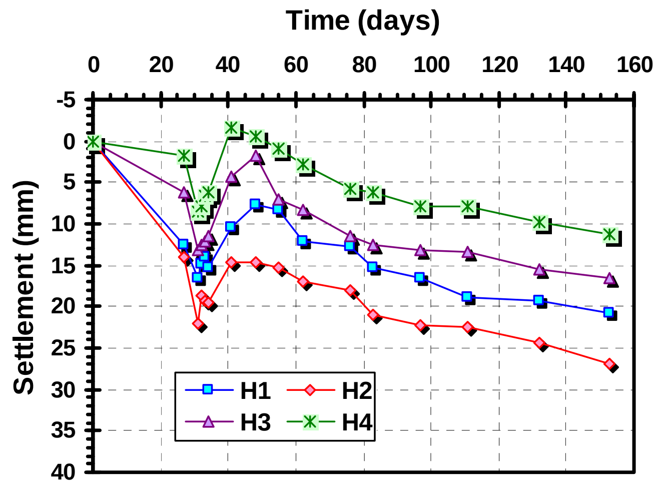

Fig. 9(h) – Settlement of block H

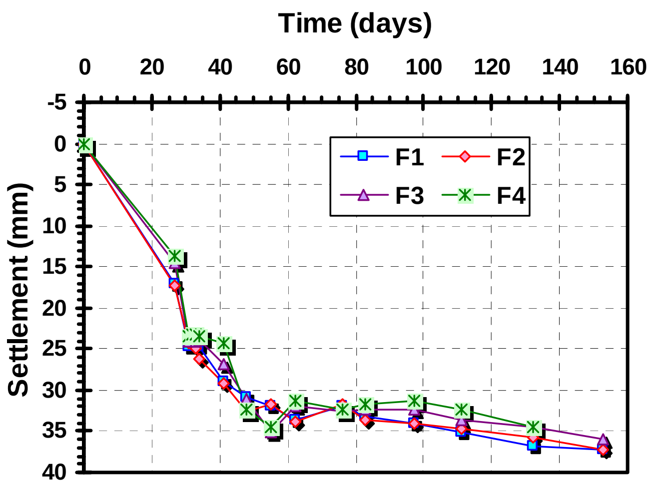

Fig. 9(f) – Settlement of block F

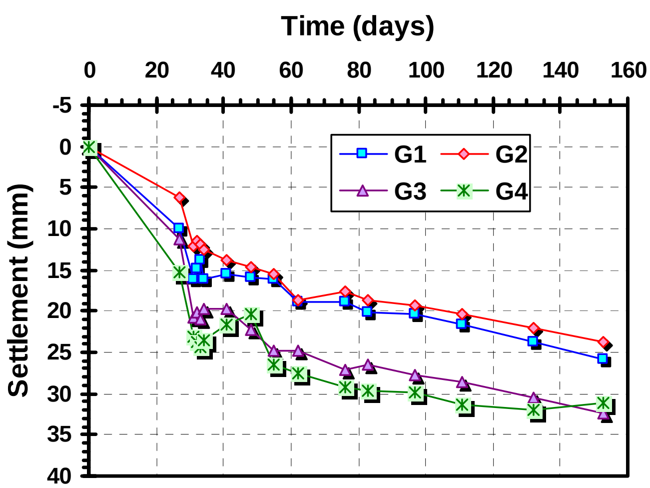

Fig. 9(g) – Settlement of block G

The average settlement for each concrete block is presented in Fig. 10. The measured settlements for concrete blocks B, C, F, G are relatively higher than for the other blocks. One possible reason is that the stresses below them are more significantly affected by the load exerted on the nearby concrete blocks. This effect was not considered in the previous settlement analysis. The maximum average settlement is about 37 mm, which occurred at concrete block F. Although the measured average settlements (16 mm to 37 mm) were larger than those calculated (15 mm), they had no adverse effects on the performance of the foundation.

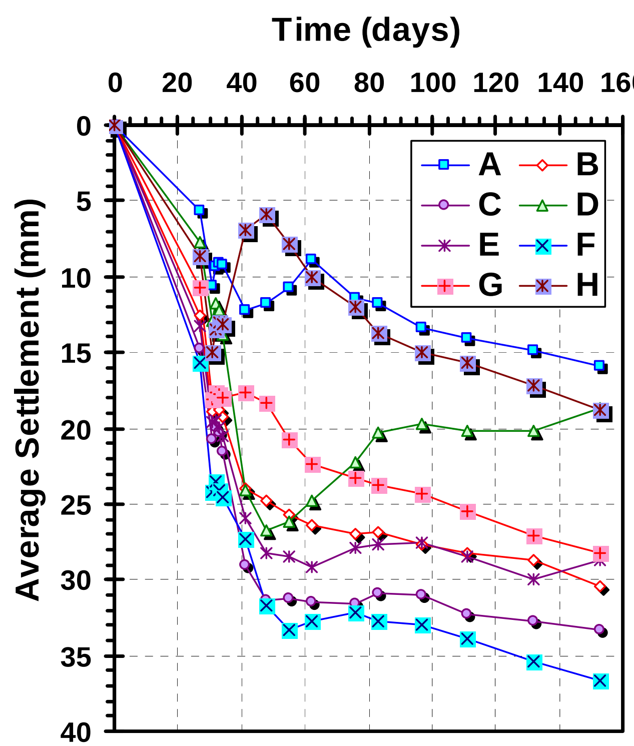

Fig. 10. Average settlement for the concrete blocks A through H

CONCLUSIONS

Due to the poor subsurface conditions at this temporary storage facility, geogrid-reinforced foundations were used to support the heavy equipment. It was found that the design shown in Fig. 6 satisfied the requirement of bearing capacity. Using elastic theory, the estimated settlement ranged from 13 mm to 15 mm. Upon completion of the foundation, the steam generators were moved into the temporary storage facility and were supported by the concrete blocks. Settlement monitoring of the concrete blocks proceeded for more than five months and disclosed settlements ranging from 16 mm to 37 mm. Even thought these settlements were larger than those calculated, the foundations performed satisfactorily.

Experience from this project shows that the use of geogrid-reinforced structural fill mats can be cost-effective and technically sound.

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