

Bearing capacity assessment of shallow foundations on hydraulic fill overlying soft marine clay.

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Abstract

Hydraulically reclaimed coastal land is increasingly used for urban development, yet shallow foundation design on hydraulic fill overlying soft marine clay remains geotechnically challenging. This study assesses the bearing capacity of shallow foundations on such a stratified profile at a reclaimed coastal site along the Lagos shoreline, Nigeria. Borehole logs, Standard Penetration Test (SPT) records and Cone Penetration Test with pore pressure measurement (CPT/CPTu) profiles were jointly interpreted to establish the subsurface stratigraphy and derive representative design parameters. The profile comprised clean to silty sand fill from the surface to approximately 6.8 m, a transitional zone to 9.5 m, clay to silty clay between 9.5 m and 12.4 m, and denser sand below 12.4 m. Adopted parameters included a fill friction angle of 35°, fill unit weight of 18.0 kN/m³, fill thickness of 7.0 m, marine clay unit weight of 16.0 kN/m³ and undrained shear strength of 20.0 kPa. Five bearing-capacity frameworks were evaluated: the single-layer methods of Terzaghi, Meyerhof, Hansen and Vesic, and the Meyerhof and Hanna (1978) two-layer punching shear model. For a 1.5 m wide strip footing at 1.0 m depth, single-layer methods yielded allowable capacities of 44.0–49.4 kPa, while the two-layer model gave 367.0 kPa. For a 10 m × 15 m raft, single-layer checks gave 46.3–51.6 kPa and the two-layer model gave 235.2 kPa. Single-layer methods are overly conservative for this stratified profile because they neglect load transfer and punching resistance through the fill. The Meyerhof and Hanna two-layer framework is therefore recommended as the more physically representative model, provided that consolidation settlement of the underlying marine clay is assessed separately.

Keywords: bearing capacity; hydraulic fill; soft marine clay; reclaimed land; Meyerhof and Hanna

Introduction

Rapid urbanisation in coastal cities has increased the demand for developable land, leading to the widespread adoption of land reclamation as an urban expansion strategy. In coastal environments, hydraulic placement of dredged sand is widely used because it enables large-scale land creation using readily available offshore or nearshore borrow materials. However, reclaimed land formed by hydraulic filling often presents complex geotechnical conditions because the placed fill is spatially variable and may overlie weak, compressible marine clay. Shallow foundation design on hydraulic fill over soft marine clay is especially difficult because the ground profile is not homogeneous. The upper fill may provide relatively high frictional resistance, but the underlying marine clay may control ultimate bearing capacity, settlement or both. In such two-layer profiles, foundation failure may occur by general shear within the fill, punching through the fill into the clay, or by a combination of both mechanisms. This makes the direct use of classical homogeneous soil bearing-capacity equations potentially unreliable.

The Lagos coastline in Nigeria provides a relevant context for this problem. Coastal reclamation has become increasingly important in Lagos because of population growth, urban expansion and

limited buildable land. The typical reclaimed profile consists of hydraulically placed sand fill overlying soft to very soft marine clay. The engineering question is therefore not only whether the reclaimed fill can support shallow foundations, but which analytical framework provides a defensible estimate of bearing capacity for this layered system. Classical bearing-capacity methods such as those proposed by Terzaghi, Meyerhof, Hansen and Vesic were developed primarily for homogeneous soil profiles. Although these methods remain useful as baseline or conservative checks, they do not explicitly model punching through a strong upper layer into a weaker lower layer. The Meyerhof and Hanna two-layer punching shear model is theoretically more appropriate for strong-over-weak soil profiles, but its practical reliability must be examined using site-specific field data.

This study converts site investigation data from a Lagos coastal reclamation site into a systematic bearing-capacity assessment of shallow foundations on hydraulic fill over soft marine clay. The specific objectives are to characterise the ground profile using borehole logs, SPT and CPT/CPTu results; derive representative soil parameters for the hydraulic fill and soft marine clay; apply selected single-layer and two-layer bearing-capacity models; compare the resulting ultimate and allowable bearing capacities; and identify the most suitable framework for design practice in similar reclaimed coastal environments.

Literature Review

Land reclamation and hydraulic fill

Land reclamation has become increasingly important in coastal cities where urban growth is constrained by limited natural land. Common reclamation techniques include dry mechanical filling, bunded reclamation, polderisation, surcharge loading, prefabricated vertical drain preloading, vacuum consolidation and rock filling. Among these methods, hydraulic filling is widely used for large-scale coastal reclamation because dredged sediments can be transported as slurry through pipelines and placed efficiently over broad areas (Arulrajah *et al.*, 2004; Chu *et al.*, 2000; Dong *et al.*, 2021). Despite its construction advantages, hydraulic fill is geotechnically variable. During slurry deposition, more coarse particles tend to settle close to the discharge point, while the fine particles are transported farther away, producing spatial variation in gradation, density and stiffness (Lee *et al.*, 1999). Where hydraulic fill is not adequately improved, it may remain loose to medium dense and may therefore be susceptible to excessive settlement, liquefaction potential or bearing-capacity failure (Chow *et al.*, 1994). In addition, the interface between the fill and underlying clay is rarely a perfect sharp boundary. Fine fill particles may infiltrate the upper clay surface, while hydraulic placement may remould the clay, creating a gradational transition zone that complicates the assumptions of ideal two-layer analytical models (Akinlabi and Adeyemi, 2021).

In the Lagos coastal environment, reclamation has been used extensively to support urban expansion. The typical reclaimed ground profile consists of hydraulically placed sand fill overlying soft to very soft high-plasticity marine clay, with denser sand or sandy clay occurring at greater depth (Oyedirán and Famakinwa, 2015). Although several studies have reported geotechnical properties of Lagos coastal soils, systematic comparison of shallow foundation bearing-capacity frameworks for hydraulic fill overlying soft marine clay remains limited.

Engineering behaviour of soft marine clay

Marine clay is a fine-grained cohesive deposit formed in low-energy offshore, estuarine or shallow coastal environments. It is commonly classified as high-plasticity clay, with high natural water

content, high void ratio and low undrained shear strength (Bjerrum, 1972; Zhu *et al.*, 2019). For West African coastal marine clays, liquid limits commonly range from about 60% to 120%, while plasticity indices may range from about 30% to 80% (Oyediran and Famakinwa, 2015; Al-Bared and Marto, 2017). The undrained shear strength of soft marine clay is a key parameter for short-term bearing-capacity assessment. Values in the upper soft clay stratum are commonly low, often within the range of 5-30 kPa (Bjerrum, 1972). High compressibility also makes consolidation settlement a major serviceability concern under foundation loading. Therefore, even where ultimate bearing capacity appears adequate, settlement may still govern foundation design. Lagos marine clay is typically described as normally consolidated to lightly overconsolidated dark grey to black silty clay of high plasticity. It often has natural water content close to or above the liquid limit and may contain organic material associated with estuarine deposition (Oyediran and Famakinwa, 2015). CPT investigations in Lagos coastal soils have reported low cone resistance in the upper clay stratum, while SPT N-values are frequently below 5, and in very soft zones may be as low as 0-2. These characteristics confirm the need for careful treatment of the underlying marine clay in shallow foundation assessment.

Bearing-capacity frameworks for shallow foundations

Bearing capacity refers to the maximum pressure that foundation soil can sustain before shear failure. Classical bearing-capacity theory is commonly expressed in the general form:

$$q_u = c'N_c + qN_q + 0.5\gamma BN_\gamma$$

where q_u is ultimate bearing capacity, c' is effective cohesion, q is surcharge at foundation level, γ is unit weight, B is foundation width, and N_c , N_q and N_γ are bearing-capacity factors. Terzaghi (1943) developed the earliest widely used classical formulation for strip footings on homogeneous soil. Meyerhof (1963) later introduced additional correction factors for shape, depth and load inclination. Hansen (1970) and Vesic (1973) further refined the correction factors and bearing-capacity factor expressions. These classical methods remain central to geotechnical practice, but they are fundamentally single-layer frameworks. They assume that the soil under the foundation is sufficiently uniform for the failure mechanism to develop within one material. This assumption is not always valid for reclaimed land where a relatively strong sand fill overlies weak marine clay. If the fill layer is thick relative to the footing width, failure may remain within the fill. However, where the fill thickness is small relative to foundation width, the foundation load may punch through the upper layer and mobilise the weaker clay below.

For strong-over-weak two-layer systems, Meyerhof and Hanna (1978) proposed a punching shear model in which the upper layer contributes side shear resistance while the lower weak layer provides basal bearing resistance. The model expresses ultimate capacity as a function of the lower-layer bearing capacity, the effective thickness of the upper layer, the footing width, the internal friction angle of the upper layer and a punching shear coefficient. The method is more physically representative for hydraulic fill over soft clay than homogeneous single-layer equations because it explicitly includes the interaction between the strong fill and weak clay. However, its original formulation assumes clearer layer boundaries than may occur in hydraulic reclamation deposits, where gradational interfaces and spatial variability are common.

CPT/CPTu and SPT for soil characterisation

In-situ testing provides a practical basis for parameter derivation in reclaimed coastal ground. The Cone Penetration Test measures cone tip resistance and sleeve friction continuously with depth, while the piezocone version additionally measures pore water pressure. CPT/CPTu is particularly

useful in hydraulic fill and soft clay because it provides high-resolution stratigraphic information and can identify abrupt or gradual transitions that may be missed by discrete sampling.

The corrected cone resistance is commonly expressed as:

$$q_t = q_c + (1 - a)u_2$$

where q_c is measured cone resistance, a is the cone net area ratio and u_2 is pore pressure measured behind the cone tip. In soft clay, undrained shear strength may be estimated using:

$$s_u = (q_t - \sigma_v0)/N_{kt}$$

where σ_v0 is total vertical stress and N_{kt} is an empirical cone factor. For normally consolidated marine clay, N_{kt} values around 14-16 are often adopted where site-specific calibration is not available (Lunne *et al.*, 1997; Robertson and Cabal, 2015).

The Standard Penetration Test remains widely used in Nigeria and West Africa because of its availability, low cost and compatibility with conventional borehole investigation. SPT blow counts are corrected to N60 and, for granular soils, may also be corrected for overburden to obtain $(N1)_{60}$. These corrected values can be used to estimate relative density and friction angle in sand. In clay, SPT-based undrained shear strength estimates are more approximate, and CPTu-derived values are generally preferred where available. In this study, SPT records serve as an important cross-check on CPT/CPTu interpretations and borehole descriptions.

Previous studies and research gap

International studies on reclaimed and layered ground have shown that hydraulic fill is often spatially heterogeneous and that bearing mechanisms in strong-over-weak profiles differ from those assumed in homogeneous soil theory. Studies on two-layer bearing capacity have also shown that the Meyerhof and Hanna approach performs reasonably for idealised sharp-boundary profiles, but its reliability may reduce where the layer transition is gradational. Within Nigeria and West Africa, available studies have described Lagos coastal soils, CPT and SPT responses, marine clay properties and geotechnical implications of land reclamation. However, the thesis material indicates that no published study has systematically compared Terzaghi, Meyerhof, Hansen, Vesic and Meyerhof-Hanna bearing-capacity frameworks using integrated borehole, CPT/CPTu and SPT data for the specific profile of hydraulic fill overlying soft marine clay in the Lagos reclaimed-land context. This gap provides the basis for the present manuscript.

Materials and Methods

Study area and geotechnical setting

The study is based on a major reclaimed coastal site along the Lagos shoreline, Nigeria. The commercial identity of the site is withheld because of data confidentiality. The site represents a hydraulically reclaimed land system in which dredged sandy material was placed over naturally deposited soft marine clay. The regional ground conditions are typical of coastal Lagos, where Quaternary coastal, lagoonal and estuarine sediments occur below reclaimed fill. The engineering profile established from the investigation consists of an upper hydraulic sand fill, an intermediate heterogeneous transition zone, an underlying soft marine clay layer and a deeper dense sand stratum. This profile represents a strong-over-weak ground system that is relevant to shallow foundation bearing-capacity assessment.

Data sources

The study used existing geotechnical investigation data obtained from the site. The dataset comprised borehole logs with visual soil descriptions and layer boundary information; SPT records

from four boreholes; and CPT/CPTu profiles from two cone soundings, identified as CPT-5 and CPT-6. The borehole logs were used to verify soil descriptions and identify major stratigraphic boundaries. SPT records were used to assess relative density in the granular fill and consistency of the clayey layers. CPT/CPTu results provided continuous profiles of cone resistance, sleeve friction, pore pressure response and soil behaviour type classification.

Interpretation of SPT data

SPT records from four boreholes were reviewed to characterise the vertical variation in soil density and consistency. The upper portions of the boreholes generally showed loose to medium dense sand, with local dense zones. Below the upper granular layer, soft to very soft sandy clay was encountered in several boreholes, followed by very dense sand at greater depth. Borehole 1 show loose to medium dense, fine to coarse sand from 1.5 m to 9.0 m, soft dark grey sandy clay from about 10.5 m to 13.5 m, and very dense sand at 15.0 m. Borehole 2 indicated medium dense sand from 1.5 m to 7.5 m, very soft to soft sandy clay from about 9.0 m to 13.5 m, and very dense sand at 15.0 m. Borehole 3 showed a more variable upper sand profile, including dense sand at 1.5 m, loose to medium dense sand between 3.0 m and 10.5 m, soft to very soft sandy clay from 12.0 m to 13.5 m, and very dense sand at 15.0 m. Borehole 4 indicated medium dense sand from 1.5 m to 10.5 m, soft sandy clay between 12.0 m and 13.5 m, and very dense sand at 15.0 m. The SPT evidence therefore confirms a stratified profile comprising an upper granular zone, a weaker clayey zone and a deeper competent sandy layer.

Interpretation of CPT/CPTu data

The CPT/CPTu interpretation focused on CPT-5 and CPT-6. Both soundings showed broadly similar profiles, although CPT-5 generally indicated higher cone resistance and higher estimated competence than CPT-6 in some intervals. The upper and intermediate depths were dominated by sand and silty sand, while lower depths showed increased clayey behaviour and higher pore pressure response. For CPT-5, the upper 0.0-3.5 m interval was interpreted as predominantly sand and silty sand. Between 3.5 m and 8.5 m, sand and silty sand occurred with occasional silty sand and sandy silt. A denser sand or sand-rich zone with higher cone resistance occurred between about 8.5 m and 10.3 m. Between about 10.3 m and 13.2 m, the profile changed to clay, clayey silt and silty clay, before returning to denser sand or silty sand near 13.2-13.5 m. For CPT-6, clean sand to silty sand was interpreted from 0.0 m to about 4.5 m. Sand to silty sand with occasional sandy silt occurred from 4.5 m to 8.0 m. A competent dense sand or sand-rich bearing zone occurred between about 8.0 m and 10.0 m. Clay, clayey silt and silty clay were interpreted from about 10.0 m to 13.0 m, followed by dense sand or silty sand at the base of the sounding.

Table 1. Integrated CPT-SPT soil profile interpretation

Approximate depth (m)	Interpreted soil behaviour	SPT comparison	Engineering significance
0.0-6.8	Predominantly clean sand to silty sand	Upper SPT section indicates loose to medium dense to dense sand	Generally favourable shallow founding material compared with the clay zone below
6.8-8.0	Transitional silty sand, sandy silt and thin clayey interbeds	SPT around 7.5 m still indicates dense sand	Heterogeneous transition zone with local variability
8.0-9.5	Sand to silty sand with local loose pockets	SPT around 9.0 m records loose sand with occasional clay	Possible weaker transition horizon
9.5-12.4	Clay to silty clay or	SPT between 10.5 m and	Critical compressible layer

	clayey silt	12.0 m records very soft to firm sandy clay	likely to control settlement
12.4-13.2+	Dense sand or sand with silt	Deeper SPT lithology also returns to sand	Relatively stronger bearing stratum at depth

Soil parameters adopted for bearing-capacity analysis

Representative design parameters were selected from the borehole, CPT/CPTu and SPT interpretations. The adopted values are presented in Table 2.

Table 2. Adopted geotechnical design parameters

Layer or foundation item	Parameter	Symbol	Adopted value
Hydraulic fill	Unit weight	γ_1	18.0 kN/m ³
Hydraulic fill	Effective cohesion	c_1'	0 kPa
Hydraulic fill	Effective friction angle	ϕ_1'	35°
Hydraulic fill	Total fill thickness	H _{fill}	7.0 m
Soft marine clay	Unit weight	γ_2	16.0 kN/m ³
Soft marine clay	Undrained shear strength	$su = c_2$	20.0 kPa
Soft marine clay	Undrained friction angle	ϕ_2	0°
Foundation	Embedment depth	D _f	1.0 m
Design check	Factor of safety	FoS	3.0

Two foundation geometries were considered. The first was a 1.5 m wide strip foundation founded at 1.0 m depth. The second was a 10 m × 15 m rectangular raft foundation, also founded at 1.0 m depth. For both cases, the effective thickness of fill below foundation level was:

$$Heff = H_{fill} - D_f = 7.0 - 1.0 = 6.0 \text{ m}$$

Bearing-capacity frameworks

Five bearing-capacity frameworks were applied. Four were classical single-layer methods: Terzaghi, Meyerhof, Hansen and Vesic. These methods treat the foundation soil as a homogeneous medium and were used as lower-bound or baseline checks. The fifth method was the Meyerhof and Hanna two-layer punching shear model, which explicitly considers punching through the stronger hydraulic fill into the weaker underlying clay. For the undrained single-layer checks, the clay was treated as the controlling homogeneous material using $\phi = 0^\circ$, $N_c = 5.14$, $N_q = 1.0$ and $N_\gamma = 0.0$, with suitable shape and depth factors depending on foundation geometry. The allowable bearing capacity was obtained by dividing ultimate bearing capacity by a factor of safety of 3.0. For the two-layer model, the ultimate bearing capacity was expressed in terms of the bearing capacity of the lower clay, punching resistance through the upper fill and the effective weight of the fill layer. For the strip footing, the general form was:

$$q_u = q_b + (2Heff/B)(c_1 + Ks\sigma'_{avg} \tan\phi_1) - \gamma_1 Heff$$

where q_b is the bearing capacity of the lower clay at the fill-clay interface, $Heff$ is the effective fill thickness below foundation level, B is foundation width, c_1 is cohesion of the hydraulic fill, Ks is the punching shear coefficient, σ'_{avg} is the average effective vertical stress through the fill and ϕ_1 is the effective friction angle of the fill. For the rectangular raft, punching resistance was converted to average pressure using the perimeter-based factor:

$$m = 2Heff(B + L)/(BL)$$

where B and L are the shorter and longer raft dimensions, respectively.

Results

Stratigraphy from integrated CPT and SPT interpretation

The combined CPT and SPT evidence confirms that the site is not a homogeneous deposit. The upper profile is mainly granular and generally favourable for shallow foundation support, but its

density varies with depth and location. The zone between about 6.8 m and 9.5 m is transitional and heterogeneous. A weaker clay-dominated interval occurs between about 9.5 m and 12.4 m. Below this, the profile returns to denser sand or sand with silt. This stratification is important because the hydraulic fill may provide substantial shear resistance and load-spreading capacity, while the soft marine clay may govern long-term settlement and, where the fill is relatively thin compared with foundation width, may influence ultimate bearing capacity through punching failure.

Selection of punching shear coefficient K_s from Meyerhof and Hanna chart

The punching shear coefficient, K_s , was selected from the Meyerhof and Hanna design chart using the adopted fill friction angle and bearing-capacity ratio. For the raft calculation, the fill-only capacity was calculated as $q_1 = 3,942.9 \text{ kPa}$, while the bearing capacity of the underlying clay at the fill-clay interface was $q_b = q_2 = 228.8 \text{ kPa}$. Therefore:

$$q_2/q_1 = 228.8/3,942.9 = 0.058 \approx 0.06$$

At $\phi_1 = 35^\circ$ and $q_2/q_1 \approx 0.06$, the chart gives an adopted conservative punching shear coefficient of $K_s = 5.8$. This design point is shown in Figure 1 and was used in the two-layer punching shear calculation.

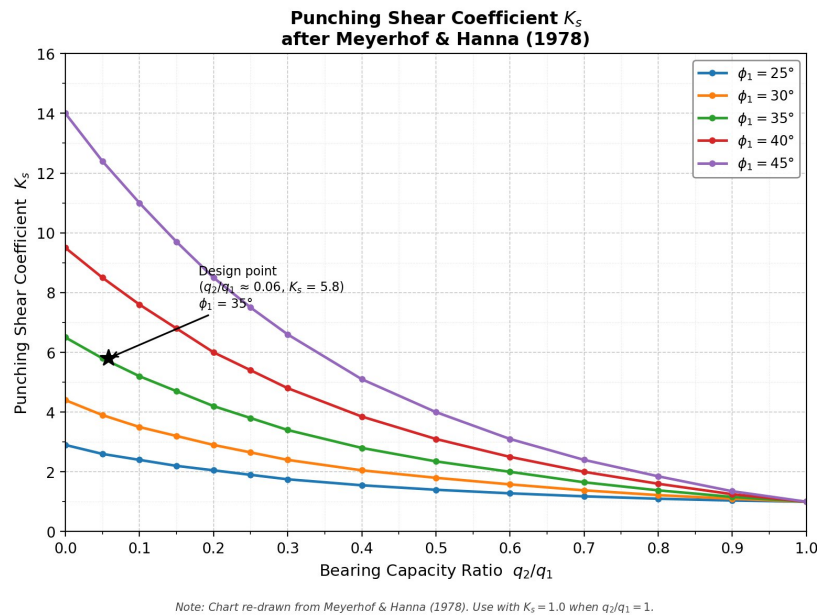


Figure 1. Punching shear coefficient K_s versus bearing capacity ratio q_2/q_1 for selected values of ϕ_1 , redrawn after Meyerhof and Hanna (1978). The marked design point corresponds to $\phi_1 = 35^\circ$, $q_2/q_1 \approx 0.06$ and $K_s = 5.8$.

Bearing-capacity results for strip foundation

For the 1.5 m wide strip foundation at 1.0 m embedment, the single-layer methods produced relatively low ultimate and allowable bearing capacities. Terzaghi's method gave an ultimate bearing capacity of 132.0 kPa and an allowable bearing capacity of 44.0 kPa. Meyerhof, Hansen and Vesic produced identical results under the adopted undrained strip-footing assumptions, giving ultimate bearing capacities of 148.2 kPa and allowable capacities of 49.4 kPa. The Meyerhof and Hanna two-layer model gave a markedly higher value. The bearing capacity of the underlying clay at the fill-clay interface was calculated as 228.8 kPa. The fill-only capacity check gave 1,100.9 kPa. The uncapped two-layer value exceeded the fill-only capacity, so the final

ultimate bearing capacity was capped at 1,100.9 kPa. With a factor of safety of 3.0, the allowable bearing capacity was 367.0 kPa.

Table 3. Bearing-capacity estimates for 1.5 m strip foundation

Method	Category	Ultimate bearing capacity, q_u (kPa)	Allowable bearing capacity, q_a (kPa), FoS = 3.0
Terzaghi (1943)	Single-layer lower-bound check	132.0	44.0
Meyerhof (1963)	Single-layer lower-bound check	148.2	49.4
Hansen (1970)	Single-layer lower-bound check	148.2	49.4
Vesic (1973)	Single-layer lower-bound check	148.2	49.4
Meyerhof and Hanna (1978)	Two-layer adopted framework	1,100.9	367.0

Bearing-capacity results for rectangular raft foundation

For the 10 m × 15 m rectangular raft, the effective fill thickness ratio was $H_{eff}/B = 6.0/10.0 = 0.6$. This indicates that the fill is not thick relative to raft width; therefore, the underlying soft clay remains relevant to the bearing mechanism and the two-layer punching model should not be replaced automatically by a fill-only assumption. The single-layer lower-bound checks gave allowable capacities between 46.3 kPa and 51.6 kPa. Terzaghi's rectangular raft check gave $q_u = 154.8$ kPa and $q_a = 51.6$ kPa. Meyerhof gave $q_u = 139.2$ kPa and $q_a = 46.4$ kPa. Hansen and Vesic each gave $q_u = 138.8$ kPa and $q_a = 46.3$ kPa. The Meyerhof and Hanna rectangular raft calculation gave $q_u = 705.6$ kPa. Applying a factor of safety of 3.0 gave an allowable bearing capacity of 235.2 kPa. This value was below the fill-only capacity check, so no capping was required.

Table 4. Bearing-capacity estimates for 10 m × 15 m rectangular raft foundation

Method	Category	Ultimate bearing capacity, q_u (kPa)	Allowable bearing capacity, q_a (kPa), FoS = 3.0
Terzaghi rectangular raft	Single-layer lower-bound check	154.8	51.6
Meyerhof rectangular raft	Single-layer lower-bound check	139.2	46.4
Hansen rectangular raft	Single-layer lower-bound check	138.8	46.3
Vesic rectangular raft	Single-layer lower-bound check	138.8	46.3
Meyerhof and Hanna rectangular raft	Two-layer adopted framework	705.6	235.2

Discussion

Influence of the layered ground profile

The results demonstrate that the bearing capacity of shallow foundations on reclaimed hydraulic fill cannot be interpreted reliably using homogeneous soil assumptions alone. The upper hydraulic fill provides frictional resistance and load spreading, but the underlying soft marine clay remains a critical component of the foundation system. The engineering behaviour depends strongly on foundation width, foundation embedment and the effective thickness of fill between the foundation

base and the clay interface. For the strip foundation, $H_{eff}/B = 6.0/1.5 = 4.0$. This indicates a thick fill layer relative to foundation width. Under this geometry, the two-layer analysis reached the fill-only capacity cap, implying that ultimate bearing resistance is governed by the hydraulic fill rather than by punching into the clay. This explains why the Meyerhof and Hanna value is much higher than the single-layer clay-based checks. For the raft foundation, $H_{eff}/B = 6.0/10.0 = 0.6$. This lower ratio means that the raft is wide relative to the effective fill thickness, so load transfer into the underlying clay is more significant. The two-layer allowable capacity remains higher than the single-layer lower-bound checks because the model still accounts for punching resistance through the fill. However, unlike the strip case, the two-layer raft result is not capped by the fill-only bearing capacity.

Comparison of single-layer and two-layer methods

The single-layer methods gave low values because they idealised the profile as homogeneous undrained soft clay. This is useful as a conservative lower-bound check, but it does not represent the actual load-transfer mechanism in the reclaimed profile. In particular, the single-layer clay-based approach ignores the contribution of the 6.0 m effective hydraulic fill thickness below foundation level. The Meyerhof and Hanna method is more physically appropriate because it includes three important mechanisms: bearing support from the underlying clay, punching shear resistance through the upper fill and the effect of overburden stress within the fill. This makes it better suited to strong-over-weak ground profiles such as hydraulic fill over soft marine clay. However, the higher capacities obtained from the two-layer model should not be interpreted as automatic design capacities without serviceability checks. Soft marine clay may experience significant consolidation settlement under sustained structural loading. This study therefore supports the use of the Meyerhof and Hanna method for ultimate bearing-capacity assessment, but it does not remove the need for settlement analysis, especially for raft foundations and medium-rise structures.

Design implications for reclaimed coastal land

The findings have practical implications for foundation design on reclaimed coastal land in Lagos and similar West African environments. First, the use of homogeneous clay-based bearing-capacity checks alone may lead to overly conservative estimates and unnecessary adoption of deep foundations for light to moderate structures. Second, the use of fill-only bearing-capacity equations without checking the underlying soft clay may be unsafe where the fill thickness is small relative to foundation width. Third, a two-layer design approach provides a more balanced framework because it recognises both the strength of the hydraulic fill and the weakness of the underlying marine clay. For narrow strip footings founded on sufficiently thick hydraulic fill, bearing capacity may be governed mainly by the fill layer. For larger raft foundations, the underlying clay becomes more relevant because the foundation width is large relative to the fill thickness. The ratio H_{eff}/B is therefore a critical design parameter and should be explicitly considered in reclaimed-land foundation assessment.

Limitations of the Study

This study is based on existing site investigation records rather than a newly designed field and laboratory testing programme. The adopted parameters were derived from available borehole, SPT and CPT/CPTu interpretations and were selected as representative design values. Although this is consistent with routine geotechnical practice, site-specific laboratory calibration of undrained shear strength, compressibility and stress history would strengthen future assessments. The study

focuses on bearing capacity as an ultimate limit state. Settlement, particularly primary consolidation settlement of the soft marine clay, was outside the scope of the bearing-capacity calculations but remains essential for final foundation design. Numerical modelling was also not undertaken; therefore, the study compares analytical frameworks rather than finite-element simulations.

Conclusion

This study assessed the bearing capacity of shallow foundations on hydraulic fill overlying soft marine clay at a reclaimed coastal site in Lagos, Nigeria, using borehole logs, SPT records and CPT/CPTu interpretations. The investigation confirmed a stratified ground profile comprising an upper granular hydraulic fill, a heterogeneous transition zone, a weaker clay-dominated interval and a deeper dense sand layer. The integrated CPT-SPT interpretation showed that the upper 0.0-6.8 m interval consists predominantly of clean sand to silty sand, while the 6.8-9.5 m interval is more variable and transitional. The 9.5-12.4 m interval is clayey and compressible, representing the critical weak layer for settlement and possible load-transfer effects. Dense sand or sand with silt occurs below approximately 12.4 m. The bearing-capacity comparison showed substantial differences between single-layer and two-layer analytical methods. For a 1.5 m strip foundation, single-layer methods gave allowable bearing capacities of 44.0-49.4 kPa, while the Meyerhof and Hanna two-layer model gave 367.0 kPa. For a 10 m × 15 m rectangular raft, single-layer methods gave allowable capacities of 46.3-51.6 kPa, while the Meyerhof and Hanna model gave 235.2 kPa.

The Meyerhof and Hanna two-layer punching shear model is the most appropriate analytical framework for this hydraulic-fill-over-soft-clay profile because it explicitly represents the interaction between the stronger hydraulic fill and weaker underlying marine clay. Classical single-layer methods remain useful as conservative lower-bound checks, but they should not be used as the sole basis for design where field evidence confirms a layered reclaimed profile. For the studied site, the recommended bearing-capacity design values are 367.0 kPa allowable capacity for the 1.5 m strip foundation case and 235.2 kPa allowable capacity for the 10 m × 15 m raft foundation case, both based on a factor of safety of 3.0. Final foundation design should also include settlement assessment, particularly consolidation settlement of the soft marine clay, because serviceability may govern even where ultimate bearing capacity appears adequate.

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