

INVESTIGATION OF THE BEHAVIOR OF BOX-SHAPED DEEP FOUNDATIONS

by

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ABSTRACT

INVESTIGATION OF THE BEHAVIOR OF BOX-SHAPED DEEP FOUNDATIONS

A box-shaped deep foundation is a new type of deep foundation, which consists of a raft structurally connected to outer walls that encompass the soil. Walls and the raft together create a structure that functions much like an inverted “box” in physical terms. The walls beneath the raft are anticipated to serve as a frame to restrain lateral motion. BSDFs employ the enclosed soil core as an integrated load-transfer element. This study initially aimed to investigate the behavior of BSDFs in sand under vertical static loads through an extensive numerical analysis utilizing a 3D FEM. Two coefficients which are BCR and SRR have been developed to better interpret the load-settlement behavior of BSDFs with different geometries, soil types and interface roughness. The presence of a soil core in a BSDF adds complexity to evaluate its load-settlement mechanism. Within the scope of this study, the quasi-monolithic behavior of BSDFs has also been examined by comparing them to embedded foundations, with the assumption that embedded foundations exhibit a complete monolithic behavior. The MBR and MSR values act as indicators for assessing the extent to which a BSDF exhibits monolithic behavior. As the MBR and MSR values approach one, it signifies that the BSDF behaves increasingly like a monolithic compound body. Furthermore, a laboratory testing program was conducted to observe the load-settlement behavior and monolithic performance of model BSDFs for a validation of the numerical analyses. Secondly, an extensive numerical analysis consisting of 195 different models was carried out to compare the efficiency of BSDFs to CPRFs. Based on the efficiency concept outlined in this study, it can be inferred that BSDFs may serve as a viable alternative to CPRFs in various scenarios. The findings of this study are expected to enhance comprehension of this emerging foundation type and hold the potential to facilitate its practical implementation. The visuals that emerged within the scope of this thesis study and whose copyright was transferred to the publishing house have been used in the thesis book in accordance with the publishing policy regarding the reuse of the author's own produced texts and graphics, available on the publisher's web page.

ÖZET

KUTU ŞEKLİNDEKİ DERİN TEMELLERİN DAVRANIŞININ İNCELENMESİ

Kutu şeklindeki derin temeller, derin temellerin yenilikçi bir türü olarak değerlendirilebilir. Duvarlar ve radye birlikte fiziksel açıdan ters bir “kutu” gibi işlev gören bir yapı oluşturur. KŞDT’ler, duvarların çevrelediği zemin çekirdeğini entegre bir yük aktarma elemanı olarak kullanır. Bu çalışma öncelikle, KŞDT’lerin kumda statik düşey yükler altındaki davranışını üç boyutlu sonlu elemanlar yöntemi kullanarak kapsamlı bir sayısal analizle incelemeyi amaçlamıştır. Farklı geometrilere, zemin tiplerine ve arayüz pürüzlülüğüne sahip KŞDT’lerin yük-yer değiştirme davranışını daha iyi yorumlamak için BCR ve SRR olmak üzere iki parametre geliştirilmiştir. Zemin çekirdeğinin varlığı, KŞDT’lerin yük-yer değiştirme mekanizmasını değerlendirmeyi oldukça karmaşık hale getirir. Bu çalışma kapsamında, KŞDT’lerin ne ölçüde monolitik bir davranış sergiledikleri, yük-yer değiştirme davranışları gömülü derin temellerle karşılaştırılarak incelenmiştir. MBR ve MSR parametreleri, monolitik davranış oranını değerlendirmek için göstergeler olarak kullanılmış ve bu değerler bire yaklaştıkça KŞDT’lerin daha bütünleşik bir yapı sergilediği kabul edilmiştir. Ayrıca, bir laboratuvar test programı yürütülerek, model KŞDT’lerin ve gömülü derin temellerin yük-yer değiştirme davranışları ve monolitik performansları gözlemlenerek sayısal analizlerle karşılaştırılmıştır. İkinci olarak, KŞDT’lerin geleneksel radyeli derin temellerle verimlilik açısından karşılaştırılması amacıyla 195 farklı model içeren kapsamlı bir sayısal analiz gerçekleştirilmiştir. Bu çalışmada belirtilen verimlilik konseptine dayalı olarak, KŞDT’lerin çeşitli durumlarda geleneksel radyeli derin temellere alternatif olabileceği değerlendirilebilir. Bu tez kapsamındaki bulguların bu yeni temel türünün yük-yer değiştirme davranışının daha iyi anlaşılmasına yardımcı olarak ilerideki pratik uygulamalara yol göstermesi amaçlanmıştır. Bu tez çalışması kapsamında ortaya çıkan ve telif hakkı yayınevine devredilen görseller, yayınevinin kendi ağ sayfasında bulunan “yazarın kendi ürettiği yazı ve grafiklerin tekrar kullanımı hakkında geçerli olan yayın politikası”na uygun olarak tez kitabında kullanılmıştır.

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LIST OF SYMBOLS

A_b	Pile base area
A_s	Pile shaft area
B	Width of a foundation
c	Cohesion
c_i	Interface cohesion
C_c	Coefficient of curvature
C_u	Coefficient of uniformity
d	Pile diameter
D	Depth of a foundation
D_{50}	Median particle size
d_q, d_γ	Depth factors
D_r	Relative density
e	Void ratio
E	Young's modulus
E_{50}^{ref}	Reference Triaxial loading stiffness
E_{oed}^{ref}	Reference Oedometer stiffness
E_{ur}^{ref}	Reference Triaxial unloading stiffness
E_{50}	Triaxial loading stiffness
E_i	Initial tangent modulus
e_{max}	Maximum void ratio
e_{min}	Minimum void ratio
E_{oed}	Oedometer stiffness
E_{ur}	Triaxial unloading stiffness
F	Load capacity at experimental tests
G	Shear Modulus
G_0^{ref}	Reference Shear Modulus at very small strains
G_i	Shear modulus of the interface
G_s	Specific Gravity
K	Earth pressure coefficient

K_n	Elastic interface normal stiffness
L	Pile length
m	Rate of stress dependency
M_1	Soil mass without the foundation
M_2	Soil mass with the foundation
N_q, N_γ	Bearing capacity factors
N_q^*, N_γ^*	Bearing capacity factors for deep foundations
P_{pile}	The sum of all pile loads
P_{soil}	Load carried by the soil
P_{total}	The total load on the foundation
P	Efficiency
P_{bsdf}^{50}	Applied loads for a 50 mm settlement on a BSDF
P_{cprf}^{50}	Applied loads for a 50 mm settlement on a CPRF
P^{ref}	Reference stress
P_u	Ultimate capacity of a pile
Q_{ult}	Ultimate bearing capacity
q^{100}	Bearing pressures at the settlements of 100 mm
q^{50}	Bearing pressures at the settlements of 50 mm
q_a	Asymptote stress
q_b	Base resistance
q_{bsdf}	Bearing capacity of a BSDF
$q_{embedded}$	Bearing capacity of an embedded foundation
q_f	Failure stress
$q_{shallow}$	Bearing capacity of a shallow foundation
R_f	Failure ratio
R_{int}	The strength reduction factor
S_{bsdf}	Settlement of a BSDF
$S_{embedded}$	Settlement of an embedded foundation
S_{min}	Minimum pile spacing
S_{pr}	Maximum settlement of the piled raft
S_q, S_γ	Shape factors
S_r	Maximum settlement of the corresponding unpiled raft
$S_{shallow}$	Settlement of a shallow foundation

t	Wall thickness
T	Raft thickness
t_i	Virtual thickness of the interface
$u_{x,y,z}$	Displacements in x,y or z directions
V_{ult}	Ultimate load
V	Total volume
V_r	Raft volume
V_w	Wall volume
α_{pr}	Piled raft coefficient
α_{pr}	Piled raft coefficient
γ_{sat}	Saturated unit weight
γ_{unsat}	Unsaturated unit weight
γ	Unit weight of the soil
$\gamma^{0.7}$	Strain level at which G_0 is 70% reduced
γ_d	Dry unit weight of the soil
γ_w	Unit weight of the water
δ	Friction angle between the wall and the soil
ε	Strain level
κ	Box factor
ν	Poisson's ratio
ν_{ur}	Poisson's ratio for unloading and reloading
ξ_s	Maximum settlement coefficient
$\xi_{\Delta s}$	Differential settlement coefficient
ρ_d	Dry density of the soil
σ'_z	Vertical effective stress
σ_1	Major principal stress
σ_3	Minor principal stress
τ_s	Shaft resistance
φ	Friction angle
φ_i	Interface friction angle
φ'	Effective friction angle
ψ	Dilatancy angle

LIST OF ACRONYMS/ABBREVIATIONS

3D	Three Dimensional
BCE	Bearing Capacity Enhancement
BCR	Bearing Capacity Ratio
BSDF	Box-shaped Deep Foundation
CPRF	Conventional Piled Raft Foundation
EC	Eurocode
FEM	Finite Element Method
HSS	Hardening Soil model with small strains
LVDT	Linear Variable Displacement Transducer
MBR	Monolithic Bearing Ratio
MSR	Monolithic Settlement Ratio
SLS	Serviceability Limit State
SRR	Settlement Reduction Ratio
ULS	Ultimate Limit State

1. INTRODUCTION

A foundation serves as a medium connecting the superstructure loads to the ground, while also transmitting any ground deformations to the superstructure. Foundation engineering involves applying the core principles of geotechnical engineering to design the foundations of diverse structures. Foundations, typically, can be classified into two categories: shallow and deep foundations. The load-bearing capacity of soils typically rises with greater depth, making deep foundations more commonly employed for larger and heavier structures, particularly in cases where poor soil conditions necessitate additional precautions.

A box-shaped deep foundation is a new type of deep foundation, which consists of a raft structurally connected to outer walls that encompass the soil, shown in Figure 1.1. Walls and the raft together create a structure that functions much like an inverted “box” in physical terms. BSDFs can be square, rectangular, and circular or any shape, and single or multiple boxes can be formed under the same raft.

BSDFs are alternatively known as rectangular closed diaphragm walls, lattice-shaped diaphragm walls, or structural cell foundations, in several applications. In the literature, various other types of pot-shaped foundations are mentioned, including skirted foundations, suction caissons, bucket foundations, and mudmat foundations. Unlike BSDFs, skirted foundations, which are predominantly shallow foundations, typically have an aspect ratio (D/B) of less than two. In contrast, foundations with higher aspect ratios, which typically range from two to eight, are often referred to as caissons. These caissons are precast structures embedded into the soil under their own weight, commonly employed for offshore structures. One of the aims of this study is to unify all pot-shaped deep foundations with different names under the term of “BSDF” and reveal a common adoption.

BSDFs employ the enclosed soil core as an integrated load-transfer element. The walls beneath the raft are anticipated to serve as a frame to restrain lateral motion. This frame can be formed by using sheet piles, barrettes-diaphragm walls, or drilled piles. Sheet piles provide lateral resistance but do not effectively transfer vertical loads. Conversely,

diaphragm walls or bored piles can substantially enhance the load-bearing capacity while also offering lateral resistance. When employing a tangent pile installation method, it is necessary to reinforce the gaps between the piles through jet-grouting to establish a closed frame. Diaphragm walls excel in transferring structural shear forces more effectively than bored piles, although walls consisting of piles exhibit greater shaft friction.

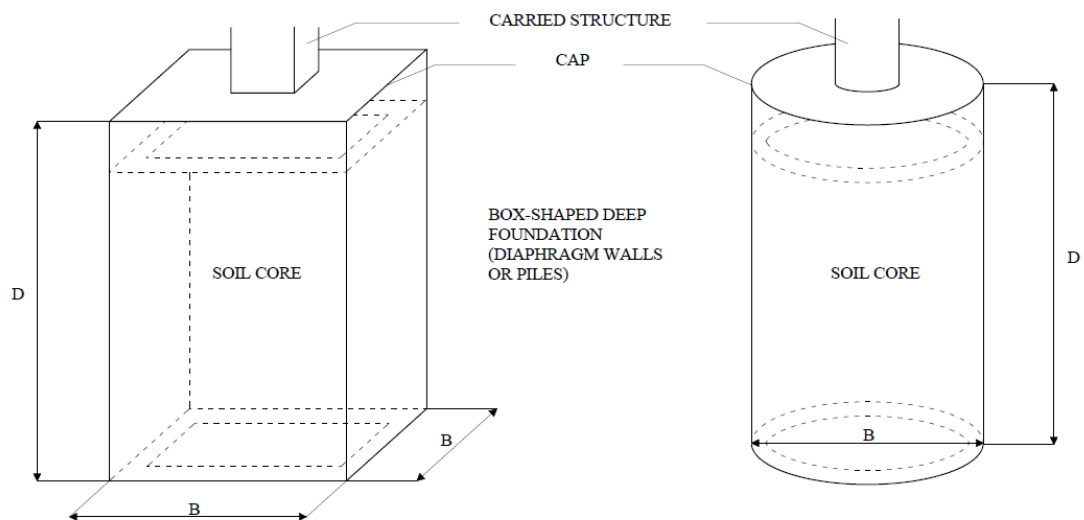


Figure 1.1. General view of BSDFs.

In the context of this thesis, Conventional Piled Raft Foundations, which are notably preferred in modern high-rise construction, will be subjected to a comparative analysis with BSDFs. CPRFs are described as geotechnical composite structures consisting of three key elements: piles, raft, and subsoil. CPRFs consider the contribution of the raft to the overall bearing capacity. In the case of BSDFs, the contribution of the raft is likewise taken into consideration. It is important to emphasize that the interactions among the box wall and raft, surrounding walls, raft and soil core, box wall and soil core, as well as shaft-base resistances collectively contribute to the intricate behavior observed in BSDFs. Furthermore, the presence of a soil core in a BSDF adds complexity to evaluate its load-settlement mechanism. Two approaches have been proposed to assess to what extent a BSDF behaves as a compound body. The monolithic approach reflects the complete composite interaction between the soil core and the structural elements. The single-element approach evaluates shaft resistance on both the external and internal surfaces of the foundation. Additionally, the tip resistance of piles and the bearing capacity of the raft are computed individually. To

sum up, BSDFs exhibit a unique behavior that falls between these two approaches which can be described as quasi-monolithic behavior. Within the scope of this study, the quasi-monolithic behavior of BSDFs has also been examined by comparing them to embedded foundations, with the assumption that embedded foundations exhibit a complete monolithic behavior.

This study initially aimed to investigate the behavior of BSDFs in sand under vertical static loads through an extensive numerical analysis utilizing a 3D FEM. To achieve this goal, a total of 339 numerical simulations were executed, aiming to investigate the load-settlement and monolithic behavior of BSDFs with varying geometries, soil-structure interface roughness, soil types and load levels. Additionally, a laboratory testing program was conducted to observe the load-settlement behavior and monolithic performance of model BSDFs. These findings were then compared to the results obtained from the numerical analysis for a validation. Secondly, an extensive numerical analysis consisting of 195 different models was carried out to compare the efficiency of BSDFs to CPRFs.

Box-shaped deep foundations have demonstrated their effectiveness in transmitting structural loads into the soil and mitigating excessive settlements. Consequently, they have undergone significant advancements, particularly in the context of bridge foundations. Nonetheless, there is limited research on this subject, and further investigation is needed to explore their efficiency for various other types of structures. The findings of this study are expected to enhance comprehension of this emerging foundation type and hold the potential to facilitate its practical implementation.

2. LITERATURE REVIEW

2.1. Previous Research on Box-Shaped Deep Foundations

Several researchers have examined the behavior of BSDFs using both physical and numerical models. Among them, a group of Austrian scholars have conducted studies on BSDFs and published several research papers. Furthermore, they have referred to the BSDF concept as an Austrian type of deep foundation.

As a pioneer, Brandl (1987) investigated the fundamentals and theory of this foundation method and its practical applications. He mentioned that assessing the bearing capacity and load transfer mechanism of BSDFs stands between the single-element approach and the monolith approach.

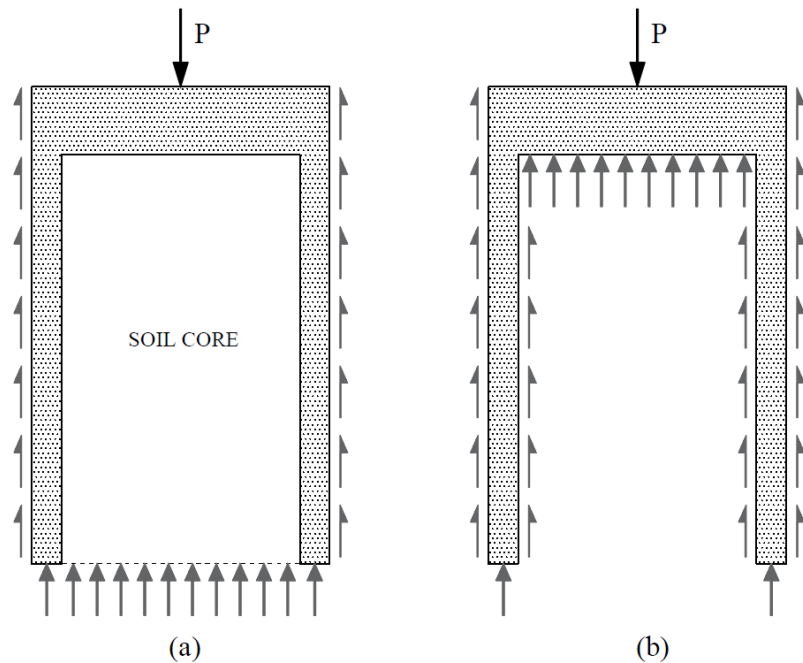


Figure 2.1. Monolithic approach (a) and single-element approach (b).

The single-element approach provides only fictitious limit values, as the bonding effect between structural elements and closed soil core is overlooked. The fact is that single

components cannot fail due to the compound effect and rigid bonding of the surrounding walls and the raft. Consequently, relatively low safety factors are adequate for this theoretical model. Another limit hypothesis is an idealized “monolith theory”. A complete bonding effect is presumed between the structural elements and the enclosed soil. For the quasi-monolith, only skin friction on the outer surface of the foundation box may be considered. Nevertheless, a full composite effect occurs only theoretically but barely in practice. This means that relatively high safety factors are necessary if the failure of equivalent shallow foundations is calculated according to conventional computational methods (Brandl, 2001).

Hofmann (2001) carried out a comprehensive model test program to study the parameters affecting the behavior of BSDFs composed of bored piles. Moreover, varying pile diameter and length, and soil density of sand were taken into consideration to investigate their impact. The study covered 70 tests with the following test series and different configurations:

- BSDFs with inner piles,
- BSDFs without inner piles,
- BSDFs without soil core,
- BSDFs filled with concrete,
- Conventional pile group,
- Single piles.

The test results showed that the efficiency of relatively small boxes is higher compared to larger boxes, although larger BSDFs carry higher loads. It is revealed that the settlement could be reduced by installing inner piles and this trend could be expressed by the cell factor. Furthermore, the boxes with cross inner walls showed a better load-settlement behavior than those with square or circular inner walls. Another parameter affecting the monolithic behavior of BSDFs was introduced as the bond factor obtained by comparing BSDFs with and without soil core. Moreover, the load-sharing mechanism between the soil core and piles was defined as a box factor. The box factor is expressed as

$$P_{\text{soil}} = \kappa P_{\text{total}}, \quad (2.1)$$

$$0 \leq \kappa \leq 0.4, \quad (2.2)$$

where P_{soil} is the load carried by the soil, P_{total} is the applied load, and κ is the box factor. Brandl and Hofmann (2002) claim that the box factor rises as the stiffness of the soil core increases and the pile length or aspect ratio decrease.

After conducting a comprehensive test program and considering in-situ measurements taken from the actual cases of BSDFs, it is claimed that the load-sharing behavior of BSDFs depends on several factors, such as:

- BSDF configuration such as varying aspect ratio, length, and width,
- Stiffness of concrete and soil,
- Load level and configuration,
- The ratio of serviceability load to ultimate load,
- Soil properties,
- Horizontal and vertical soil movement,
- Stress level and configuration between raft and soil,
- Installation affects.

Brandl (2010) cited the advantages of BSDFs by utilizing the theory, model tests, in-situ measurements, and observation of real cases:

- Increasing bearing capacity stemmed from the quasi-monolith structure,
- Reduction of total and differential settlements,
- Smaller foundation area requirement compared to CPRFs,
- High resistance against lateral forces caused by retaining structures,
- Quite appropriate for the case of highly heterogeneous soil,
- Highly resistant to scouring,
- Highly resistant to earthquake and liquefaction,
- Highly appropriate in the fluctuation of cyclic loads.

The practical applications of BSDFs have also been studied by considering EC 7-1 (Fross et al., 2010). As a case study, the bearing capacity of the BSDF under the pier of the Danube Bridge in Wien was calculated according to the monolithic approach. Namely, the skin friction on the outer surface of the BSDF and the base resistance of the equivalent

shallow foundation were reduced by the safety factors given in EC 7-1. Furthermore, Sava Bridge in Belgrade was chosen as a second case study to show the estimated settlement values of a BSDF. They concluded that all bridge foundations over the Danube, which were built as BSDFs proved to have satisfactory behavior against failure and settlement exceeding serviceability limits.

Nagy et al. (2020) carried out a numerical analysis to investigate the load-settlement behavior of the BSDF under the Sava Bridge in Belgrade. The estimated settlements were also compared to the measured results.

Wen et al. (2009) conducted a model test including a BSDF and a single diaphragm wall to study the load-settlement behavior and load-sharing mechanism of BSDFs. A disturbed loess as the test soil and two groups with different geometries of structures were chosen in model testing. The results of this study can be summarized with the following findings:

- If the surface area of the soil core under the raft is expanded to some degree, the distribution of the soil pressure under the raft will get closer to uniform.
- The inner skin friction on the upper parts of the box weakens due to limited displacement between the soil and the walls resulting from the silo pressure and the compressed soil core under the raft.
- The load taken from the soil under the raft is around 10-20% of the total load, so the raft's contribution to the load sharing should not be ignored.

Cheng et al. (2012) studied the behavior of BSDFs under horizontal and vertical loads performing numerical analysis. The most significant point of this study was to investigate the load settlement behavior and load-sharing mechanism of BSDFs under different load levels. They have reached some crucial conclusions dealing with the varying load levels. Firstly, it is concluded that the load shared by the upper parts of the inner walls is perpetually small and can be ignored for all load levels. It is only after the mobilization of outer skin friction and tip resistance that the inner skin friction mobilizes. The contribution of the soil core to the load-carrying increases with the increasing cross-sectional dimensions. As the depth of the box increases, so do the load carried by the outer walls.

Wu et al. (2015) discussed the efficiency of BSDFs by comparing them with CPRFs carrying out some model tests owning the geometric similarity ratio of 1:30 and numerical simulations. Different layers of soil consisting of silty clay, muddy clay, and sand and three foundation models including CPRFs, and BSDFs with one and two cells, respectively have been used to study the load settlement behavior and load sharing mechanism. It is said that the dimensions of model test foundations were determined in line with the principle of using a similar amount of material. However, the volumes of BSDF and CPRF are quite different since the raft volumes are not the same. Therefore, comparing BSDFs to CPRFs with different geometries and volumes raises the issue of the necessity of a comparison parameter. The numerical simulations were used to validate the model test results, and load settlement behaviors showed a reasonable agreement. It is concluded that as the wall depth increases, a greater proportion of the loading is supported by the outer skin friction, the tip resistance of the wall body increases, and the soil core takes a smaller proportion of the loading. First, the outer parts of the walls bear the greater proportion of loading. Then, as load increases, the proportion of loading supported by outer parts of the walls decreases while the proportion of loading supported by tip resistance increases. When it comes to the soil core, it almost stands still after taking its proportion of loading.

Wu et al. (2016) proposed an analytic approach to predict the settlement values of BSDFs by using the strain-gauge data from the model tests and comparing them to numerical simulations. Additionally, Wu et al. (2020) conducted a full-scale model test to validate the iterative analytic approach. The model BSDF had one cell with an outer cross-section of 3.4 x 3.4 m, a wall thickness of 0.8 m, and a depth of 15 m.

Martinez and Romo (2018) defined the BSDFs as structural cells transferring loads into the soil having a slightly higher unit weight than that of the soil it encloses. Then, they studied a real case of a BSDF carrying the pillars of a viaduct in Mexico City by performing a 3D numerical simulation. The noteworthy aspect of this study was its proposal of BSDF as an alternative solution to CPRF, which had already been accepted for the project. Ultimately, it was revealed that the uniform and differential settlements of the actual foundation matched the values obtained through numerical analysis.

Wu et al. (2017) investigated the soil arching effect for BSDFs by utilizing particle-flow-code software. Two 3D models of BSDFs were numerically simulated to verify the flow-code software results. The reason lying behind the soil arching effect for BSDFs was claimed to be the differential displacements of the lower part of the walls and the soil. The parametric study revealed that the soil arching was highly affected by the width and depth of the BSDF, and relatively high and low values of this parameter negatively affect the soil arching. Additionally, as the stiffness and strength of the soil increase, so does the soil arching effect.

2.2. Previous Research on Other Pot-Shaped Foundations on Sand

Other pot-shaped foundation types found in the literature include skirted foundations, suction caissons, bucket foundations, and mudmat foundations. Skirted foundations, unlike BSDFs, are primarily shallow foundations with an aspect ratio (D/B) smaller than two. On the other hand, caissons are foundations with larger aspect ratios, typically ranging between two and eight. These caissons are precast structures that are embedded into the soil under their weight and are commonly utilized for offshore structures.

Many researchers have proposed bearing capacity formulas by utilizing slip line theory and limit equilibrium technique (Meyerhof, 1963; Hansen, 1970; Vesic, 1973). They have claimed that the bearing capacity of a foundation depends on the properties of the soil, dimensions, shape, depth of the foundation, and eccentricity and inclination of the load. As a general form, the bearing capacity of foundations on sand is expressed as

$$Q_{ult} = \gamma D N_q s_q d_q + 0.5 \gamma B N_\gamma s_\gamma d_\gamma, \quad (2.3)$$

where Q_{ult} is the ultimate bearing capacity, γ is the unit weight of the soil, D is the depth of the foundation, N_q and N_γ refer to bearing capacity factors, s_q and s_γ are shape factors and d_q and d_γ are depth factors. Since pot-shaped foundations on sand with aspect ratios lower than two can be considered shallow foundations, some researchers dealing with pot-shaped foundations have used this formula to determine the bearing capacity (Byrne et al., 1999; Ibsen et al., 2012; Park et al., 2016)

Vesic (1963) had previously published a study on factors influencing the bearing capacity of deep foundations on sand. Large-scale model tests have shown that failure modes

of foundations depend on the relative density of sand and the aspect ratio of the foundation. All three types of failure modes, which are general shear failure, local shear failure, and punching shear failure, occur at shallow foundations. However, owing to the greater depth of deep foundations, they often fail in punching shear failure mode.

The total capacity of a single pile in sand is typically estimated as

$$P_u = q_b A_b + \tau_s A_s, \quad (2.4)$$

where P_u is total ultimate capacity, q_b and τ_s are base resistance and shaft resistance, and A_b and A_s are areas of pile base and shaft, respectively. Base resistance is $q_b = \sigma'_z N_q^* + \gamma B N_\gamma^*$ and different from the bearing capacity factors in shallow foundations, N_q^* and N_γ^* used in deep foundations depend on both compressibility and shear strength of the soil. Vesic (1977) offered the values for N_q^* and N_γ^* . Shaft resistance is $\tau_s = K \tan \delta \sigma'_z$, and δ and K are soil-foundation interface friction angle, and earth pressure coefficient on the shaft, respectively. Kulhawy (1984) and Reese and O'Neil (1989) recommended the values for δ and K after conducting several model tests.

Parallel to the development of offshore technology, pot-shaped foundations have also evolved and been greatly used. Since offshore structures are subjected to vertical, horizontal, and moment loadings, many preliminary studies have been published to investigate the behavior of pot-shaped foundations under this combined loading utilizing numerical and experimental methods. For example, the suction caisson is a type of pot foundation that is installed into the soil by pumping the water out of the pot and, as a result, obtaining a penetration force (Tjelta, 1994; Bye et al., 1995; Bransby and Randolph, 1998; Bransby and Randolph, 1999)

Byrne and Houlsby (1999) investigated the behavior of circular skirted foundations under combined loading conditions carrying out model tests in dense sand with the aspect ratios of 0, 0.16, 0.33, and 0.66. A general equation for estimating the vertical bearing capacity of a skirted foundation was suggested by utilizing the vertical bearing capacity of a surface foundation and the aspect ratio of the skirted foundation. To decrease the scatter on relative density versus bearing capacity plots of the previous study, further model tests and a series of finite element analyses were conducted to update the previous equation by several researchers (Ibsen et al., 2012).

Villalobos (2007) conducted an experimental study using skirted foundation models with aspect ratios of 0.26 to 2.01. Bearing capacity factors, N_q and N_γ , were obtained using experimental load settlement plots and analytical bearing capacity formulas presented in the literature. Bearing capacity factors were found to increase with the increasing strength and relative density of the soil and the aspect ratio of the skirted foundation.

Park et al. (2016) carried out finite element analysis utilizing Mohr-Coulomb criteria with a non-associated flow rule to determine the vertical bearing capacity and load transfer mechanism of bucket foundations on the sand with varying aspect ratios of 0.25 to two. They modeled 31 finite element models comprising three types of foundations: bucket foundation with skin resistance, embedded foundation, and bucket foundation without skin resistance. The skin friction and base resistance values of numerical analysis were compared to analytical values and the study showed that:

- The analytically calculated skin friction increases linearly with depth and is independent of the settlement. The numerically calculated skin friction depends on the settlement and increases to a certain level higher than the analytically calculated one and then decreases. The increase in skin friction is observed due to increased horizontal stress after arching. While the foundation and drifting soil go downward, the more constrained soil at the bottom part of the foundation than the soil near the top part of the foundation leads to soil arching. The increase in shaft friction due to arching effect is more pronounced for bucket foundations with low aspect ratios.
- The soil arching leads the failure surface to widen because of the increasing vertical stress in the sand. Shape and depth factors (s_q , d_q) for the bucket foundation which are back-calculated are higher than that of the embedded foundation. Park et al. (2016) cited that the difference depends on the strength parameters and aspect ratio, however, the compressibility of the soil was not included.

Al-Aghbari and Mohamedzein (2018) performed a model test study consisting of shallow circular skirted foundations with aspect ratios of 0.25 to 1.25 to observe the load-settlement behavior and to derive skirt factors by utilizing the formula given above. The sand used in the test was fine-grained, poorly graded sand with different relative densities ranging

from medium to dense. The shear box test was carried out to determine the friction angle between the raft and the soil, and the skirts and the soil. To study the effect of the interface roughness, the surfaces of the skirts were also roughened. The skirt factor was composed of two components named skin depth factor and skin surface factor and a modified version of the conventional bearing capacity equation for shallow circular-shaped skirted foundations in the sand was offered.

Katri and Kumar (2019) studied the load-settlement behavior of square and circular skirted foundations with aspect ratios of 0.25 to two utilizing a numerical analysis. The numerical upper and lower bound values of bearing capacities of skirted foundations with varying depth, different surface roughness, and soil strength were calculated and compared to values calculated using conventional bearing capacity equations. The bearing capacity ratio referring to the ratio of the bearing capacity of the skirted foundation to the shallow foundation was introduced for comparison and the following outcomes were disclosed:

- Bearing capacity ratios of circular skirted foundations were higher than that of square skirted foundations.
- Bearing capacity ratios were found to increase with a decrease in soil strength and the authors described it as an increase in efficiency.
- The foundations with higher surface roughness had higher bearing capacity values.

Eid (2013) carried out a comprehensive numerical analysis and physical tests to study the load-settlement behavior of skirted foundations on the sand with aspect ratios of 0.5 to two. Since the load settlement plots did not present a clear yielding point or peak value, the capacity load was determined according to the load corresponding to the settlement equal to 10% of the width. In addition, the extent to which parameters such as skirt depth, the cross-section of the foundation, and soil strength affect the bearing capacity and settlement was evaluated. Finally, some equations were presented to determine the bearing capacity and settlement of the skirted foundations.

Unlike the previous studies, Khatri et al. (2017) investigated the load-settlement behavior of the square and the rectangular skirted foundations. An experimental

investigation including skirted foundations with the aspect ratios of 0.25 to one was carried out and the following conclusions were drawn:

- The load settlement plots showed that clear peak pressure values could be observed for samples in the sand with the relative densities of 50%, 70%, and 87%; however, those with a relative density of 30% did not have a precise peak pressure value.
- The bearing capacity ratios of skirted foundations on looser sand were found to be higher compared to those on looser sand. Khatri et al. (2017) also described this trend as efficiency.
- Regardless of being square or rectangular, all skirted foundations showed a similar trend of increase in bearing capacity with increasing aspect ratios.

Thakur and Dutta (2018) performed a numerical and experimental analysis to assess the bearing capacity of unskirted, singly, and doubly skirted square foundations with aspect ratios of 0.5 to 1.5 on three types of sands with different relative densities. Other than findings similar to previous studies, it was worth noting that the comparison between numerical and experimental simulations with the same geometry was made.

Ghaseminejad et al. (2018) carried out a numerical analysis to investigate the influence of such variables as geometry, soil type, and interface roughness, on the bearing capacity of bucket foundations with varying aspect ratios of 0.25 to one. It was observed that the change in bearing capacity was less dependent on the diameter but much more on the depth, interface roughness, and sand types. The bucket foundations with a rough interface and denser soil led to more soil arching.

Using laboratory modeling tests, Sajjad and Masoud (2018) studied the load settlement behavior of skirted shallow foundations in the sand with aspect ratios of 0.5 to two. The influence of width, depth, aspect ratio, soil strength, and interface roughness on the bearing capacity and the settlement were investigated. The test results showed that the bearing capacity increase in loose soil was higher than that in dense sand. The reason behind this phenomenon was explained as more space was available to increase the shear strength of loose soil than dense soil. Namely, the bearing capacity ratio is higher in loose soil because

the displacement to mobilize the strength in dense soil is not enough compared to that in loose soil.

Haddad and Amini (2019) conducted model tests and numerical analysis to evaluate the ultimate vertical load capacity of bucket foundations in loose sand with aspect ratios of 0.5 and one. In addition, a depth factor was offered to estimate the vertical load capacity of bucket foundations by utilizing the friction angle and the aspect ratio.

Gnananandarao et al. (2020) investigated the load-settlement behavior of plus and double-box-shaped skirted foundations with aspect ratios of 0.25 to 1.5 by performing a model test. The effect of geometry, aspect ratio, relative density, and interface roughness was evaluated and the settlement reduction factor was suggested to estimate the settlement of the skirted foundation.

2.3. Previous Research on Conventional Piled Raft Foundations

Piled rafts are defined as geotechnical compound structures combined of three elements: piles, raft, and soil, which consider the contribution of the raft to the overall bearing capacity. In BSDFs, the assistance of the raft is also considered, similar to CPRFs. A complex behavior occurs because of the interaction between structural elements and the soil, including soil-pile, soil-raft, raft-pile, and pile-pile interactions.

Like any other foundations, CPRFs also require some design issues which should be taken into consideration:

- Ultimate vertical, horizontal and moment load capacities,
- The uniform settlement,
- The differential settlement,
- The structural design of the raft and the piles by considering the shear force and the moment.

Randolph (1994) stated that in a design concept, the settlement of structures should be considered first, as opposed to what design methods usually suggest to consider bearing

capacity first. The reason behind this statement is that the requirements of the serviceability limit design might be more critical than that of the limit state design.

Poulos (2001) recommended a design process including four stages for piled raft foundations:

- An initial stage to evaluate the necessity of a CPRF,
- Determining the required number of piles satisfying the requirements of the design and the general characteristics of the piles and their location,
- A detailed final stage to decide on the optimum number, location, and configuration of the piles,
- Calculating the settlement and internal forces of the structural elements including piles and the raft.

Poulos (2001) also summarized the methods for analyzing the piled raft foundations in the literature:

- The methods where the raft and the piles are represented by a series of strip foundations and springs, respectively,
- The methods where the raft and the piles are represented by a plate and springs, respectively,
- The boundary element methods discretizing the raft and the piles using elastic theory,
- The combined method using the boundary element analysis for the piles and finite element analysis for the raft,
- The basic finite element approach representing the system as a plane strain or an axisymmetric problem,
- The 3D finite element approach.

Factors affecting the behavior of CPRFs and the optimum design have been examined by many researchers (Randolph, 1994; Poulos, 2001; Prakoso and Kulhawy, 2001; Reul and Randolph, 2003; Reul and Randolph, 2004; Reul, 2004; Halder and Mana, 2004) and the following factors might be studied for BSDFs as well:

- Length of piles, the width of CPRF (length of BSDF walls, the width of BSDF),
- Diameter of piles (BSDF wall thickness),
- The number of piles, pile spacing ratio, and location of piles (BSDF configuration),
- Thickness of the raft,
- Load distribution and level,
- Physical and mechanical characteristics of the soil.

Poulos (2001) figured out that increasing the number of piles benefits the behavior. Still, there is an upper limit to the number of piles, after which the additional benefit is negligible. The thickness of the raft affects the differential settlement and the internal forces, but its effect on the uniform settlement and load sharing is a little. The differential settlement can be controlled by installing a relatively small number of piles in strategic locations. The load level and distribution are important for differential settlements and the internal forces, but not much for the uniform settlement and the load-sharing mechanism between the piles and the raft.

Fellenius (2023) pointed out that determining the thickness of the raft is a part of the design and that the size is governed by the diameter of the pile and the pile spacing. As the pile spacing increases, so does the thickness of the raft, but the close spacing can lead to physical interference of the piles. The pile spacing can be selected as 2.5 to three times the pile diameter. However, considering the long piles, the minimum pile spacing can be given as

$$s_{min} = 2.5 d + 0.02 L, \quad (2.5)$$

where s_{min} is the minimum pile spacing, d and L are the diameter of the pile and the length of the pile, respectively.

Prakoso and Kulhawy (2001) conducted the parametric FEM to examine the effects of the CPRF system geometries on average and differential settlements, load-sharing mechanisms, and internal forces. As a result, a displacement-based design methodology was developed and recommended as a flowchart for CPRFs.

Reul and Randolph (2004) studied the three cases of CPRFs on overconsolidated clay using the 3D FEM and compared the numerical results to the field measurements. A

reasonable agreement between the numerical results and the field measurements was observed. Three coefficients indicating the performance of the CPRFs were introduced. The piled raft coefficient or load-sharing ratio can be expressed as

$$\alpha_{pr} = \sum P_{pile} / P_{total}, \quad (2.6)$$

where α_{pr} represents the piled raft coefficient and P_{pile} and P_{total} stand for the sum of all pile loads and the total applied load on the foundation, respectively. Secondly, the coefficient of the maximum settlement can be expressed as

$$\xi_s = s_{pr} / s_r, \quad (2.7)$$

where ξ_s , s_{pr} and s_r stand for the coefficient of the maximum settlement, the maximum settlement of the piled raft, and the maximum settlement of the corresponding unpiled raft, respectively. Thirdly, the coefficient of differential settlement, ξ_{ds} , is defined as the differential settlement between the center and the middle of the shorter side of the raft.

Reul and Randolph (2004) performed a parametric 3D FEM to assess the influence of pile configuration, the pile number, the pile length, the raft-soil stiffness ratio, the load distribution, and the load level on the load-settlement behavior of different CPRF configurations. It turned out that the optimum design depends on the soil characteristics, the load level, and distribution. Consequently, it is impossible to generalize the results of certain parametric studies to all cases in practice. However, it can be said that the differential settlements are more susceptible to the raft-soil stiffness ratio and the load configuration than the uniform settlement. Additionally, the differential settlements under uniform loading or core-edge loading can be most efficiently decreased by installing the piles only under the central part of the raft.

Reul (2004) conducted a parametric 3D FEM analysis of CPRFs with different configurations and figured out that the shaft resistance increases with an increase in load or settlement and the piles of the CPRF do not reach their ultimate bearing capacity. The numerical study showed that the maximum settlement coefficient depends on the load level and the pile configuration. It tends to increase with a minimum pile number with longer piles rather than a bigger number of piles.

Halder and Manna (2020) carried out a 3D FEM analysis to observe the influence of pile configuration, pile spacing, pile length and raft thickness on the load-settlement

behavior of CPRFs in sand subjected to vertical loading. In addition, they introduced a non-dimensional parameter named the bearing capacity enhancement (BCE) which is the ratio of the bearing capacity of the piled raft to the bearing capacity of the unpiled raft. It has turned out that BCE increases with the pile spacing and the overlapping of the pressure bulbs led to the bearing capacity of piled rafts decreasing. Furthermore, BCE decreases with the rising load level. The bearing capacity of the piled raft increases with the pile number, but the increase in raft thickness only affects the bearing capacity a little. It has also turned out that the skin friction fully mobilizes at a settlement level of 0.5% to 1% of the pile diameter. However, the base resistance gets fully mobilized at a settlement level of 10%-20% of the pile diameter.

3. METHODOLOGY

3.1. Research Objectives and Approach

This study aims to enhance the prevalent information necessary for understanding the load-settlement behavior of box-shaped deep foundations in sand under static vertical loads. The outcomes of this study will contribute to a better understanding of this emerging type of foundation and promise to facilitate its practical application.

Identifying the gap in the literature and focusing on future geotechnical developments, the more specific objectives are as follows:

- To study the load-settlement behavior and the failure mechanisms of BSDFs with different geometries and soil-structure characteristics.
- To study the load-sharing mechanism of BSDFs and the interaction between soil, enclosing structure, and raft.
- To study the monolithic behavior of BSDFs to assess the contribution of the soil core and the effects of the other parameters such as geometry, soil type, and surface roughness.
- To present factors representing the load-settlement and monolithic behavior and to provide an optimum design by changing the depth, width, aspect ratio, or configuration of BSDFs.
- To perform laboratory tests representing various BSDFs on a small scale to observe the load-settlement and monolithic behavior of BSDFs in sand.
- To compare the load-settlement behavior of BSDFs and CPRFs to assess the efficiency and advantages of both systems.
- For assessing the efficiency of BSDFs, to study the effects of different geometry and configurations of both systems with different soil types and surface roughness.
- To investigate the effects of different load configurations and levels on the behavior of BSDFs and CPRFs considering the efficiency concept.
- To provide a guideline for designers, which is essential to evaluate the necessity of a BSDF and the proper design configuration in a real case.

The work presented in this study distinguishes itself from previous research on vertically loaded BSDFs in sand in five key aspects:

- (i) Consolidating various pot-shaped deep foundation concepts under the unified term of “Box Shaped Deep Foundations” highlighting their shared characteristics.
- (ii) Encompassing BSDFs with aspect ratios higher than two, up to five, to better represent deep foundations and punching shear failure modes.
- (iii) Two new coefficients, MBR and MSR, are introduced in this study to observe the monolithic behavior of BSDFs by comparing their bearing capacities and settlements with those of embedded foundations.
- (iv) Introducing BCR and SRR coefficients to examine the load-settlement behavior of BSDFs based on different geometries, soil types, and interface roughness.
- (v) To assess the efficiency of BSDFs in comparison to piled raft foundations and determine under what circumstances they can serve as an alternative to conventional piled raft foundations.

In this research, the behavior of BSDFs under vertical static loads is investigated using an extensive numerical analysis employing a 3D FEM. A total of 339 numerical simulations were conducted to observe the load-settlement and monolithic behavior of BSDFs with varying geometries, soil-structure interface roughness, and soil types. Additionally, a laboratory testing program was implemented to validate the results of the numerical analysis and observe the load-settlement and monolithic behavior of model BSDFs. Additionally, a comprehensive numerical analysis involving 195 distinct models was conducted to evaluate the efficiency of BSDFs when compared with CPRFs.

3.2. Numerical Simulation

This numerical study evaluates the performance of BSDFs using a 3D FEM with the aid of PLAXIS 3D V20 (2019) for modeling and analysis of the foundation systems.

In recent decades, numerical analysis methods have been popular for geotechnical problems. FEM, amongst other numerical methods, provides a powerful tool for conducting analyses simulating real cases. However, selecting the proper material model and model

parameters play an important role in coming across results presenting the actual situation to a better extent.

3.2.1. Material Model

In numerical analysis, the relation between the stress and the strain of a given material is represented by constitutive models. The research in constitutive models for soil and geotechnical structures is greatly advanced, therefore various constitutive models are available for practical applications. They range from basic to very advanced models and basic models need only a limited number of parameters to define the behavior of the soil or the structural material. However, advanced models may better represent the actual behavior but require more parameters. Constitutive models mainly used for geotechnical problems can be listed from the most basic to the most advanced as the Linear Elastic model, Mohr-Coulomb model, Hardening Soil model, and Hardening Soil model with small-strain stiffness.

The choice of a constitutive model depends on many factors but is generally linked to the type of analysis the user intends to conduct. SLS analysis requires an application of advanced constitutive models such as the Hardening Soil model which predicts the stress-strain relation more accurately than simple linear-elastic, perfectly plastic models such as the Mohr-Coulomb model.

The Linear Elastic model has been selected for the structural elements in this study. The Linear Elastic model utilizes Hooke's law of isotropic elasticity and includes two elastic parameters: Young's modulus E and Poisson's ratio ν . It can be preferred as a constitutive model representing stiff volumes in the soil, such as foundations, piles, or diaphragm walls.

The linear elastic perfectly plastic Mohr-Coulomb model includes five strength and strain parameters: Young's modulus E , and Poisson's ratio ν for the elastic behavior of the soil; friction angle ϕ and cohesion c for the plastic behavior of the soil and, dilatancy angle ψ . However, since the Mohr-Coulomb model does not represent the stress-dependent stiffness behavior and irreversible strains due to primary isotropic compression, the Hardening Soil model may be preferred as an elastoplastic model with isotropic hardening.

In addition to parameters offered by the Mohr-Coulomb model, the Hardening Soil model describes the soil stiffness more accurately by utilizing three input stiffnesses: triaxial loading stiffness, E_{50} , and the triaxial unloading stiffness, E_{ur} , and the oedometer stiffness, E_{oed} . Moreover, the Hardening Soil model with the small-strain stiffness is a modified version of the Hardening Soil model that expresses the stiffness of the soil at small strains. It can be said that most soils have a higher stiffness at low strain levels that varies non-linearly with strain. Therefore, the additional strain parameters, small-strain shear modulus G_0^{ref} and the $\gamma^{0.7}$ strain level at which the small-strain shear modulus is 70% reduced, are introduced to establish this behavior.

Brinkgreve et al. (2010) claimed that for a first approximation or an early design, it is better to use an advanced constitutive model such as the Hardening Soil model with small strains than a simple model. Since most soil properties are correlated to some extent, it is argued that an analysis using an advanced model with many parameters gives a better solution with reasonable accuracy for an early design or first approximation.

Obrzud and Truty (2020) stated that the Hardening Soil model does not explain the large values of soil stiffness related to very small strain levels which are smaller than $\epsilon=10^{-3}$. Correspondingly, choosing the Hardening Soil model with the small strain could be more proper even in applications dominated by engineering strain levels ($\epsilon>10^{-3}$). Therefore, the Hardening Soil model with the small strain has been selected as the constitutive model for the soil with different relative densities in this study.

3.2.2. Determination of Soil Parameters

Two limit states which are the ultimate limit state (ULS) and the serviceability limit state (SLS) can also be described to assess the behavior of BSDFs. ULS is a condition that leads to failure if exceeded, for example, the ultimate vertical bearing capacity of a BSDF. When it comes to SLS, it represents the exceeding a limit condition in which the required function is not fulfilled, for example, the maximum allowed settlement.

An analysis conducted for ULS considerations necessitates soil strength properties, such as the friction angle for sands and the undrained shear strength for clays. Conversely, when designing for SLS, it is essential to consider stiffness parameters. Over time, researchers have extensively examined correlations between stiffness, strength, and index properties. For sands, the primary index property to determine strength and stiffness parameters is relative density (D_r), while for clays, numerous correlations exist, often based on the plasticity index. Obtaining the required parameters for the HSS can be achieved through direct laboratory and in-situ tests or indirectly by utilizing derived correlations.

Brinkgreve et al. (2010) have proposed empirical equations for sands to calculate the model parameters under drained conditions for the HSS based on relative density. These empirical equations were developed through regression analysis using a dataset comprising general soil data, triaxial test data, and oedometer test data obtained from previous research studies. To validate the accuracy of these formulas, comparisons were made with actual test data, and they were applied to real-world scenarios for deformation predictions.

The relative density (D_r) expressed as a percentage and characterized as $(e_{max}-e) / (e_{max}-e_{min})$, where e is the initial void ratio, e_{max} is the maximum or loosest packing void ratio and e_{min} is the minimum or densest packing void ratio. By utilizing the regression analysis, the unit weights of the sand can be estimated as

$$\gamma_{unsat} = 15 + 4 (D_r/100) \text{ [kN/m}^2\text{]}, \quad (3.1)$$

$$\gamma_{sat} = 19 + 1.6 (D_r/100) \text{ [kN/m}^2\text{]}. \quad (3.2)$$

HSS includes four stiffness parameters. Brinkgreve et al. (2010) offered the following formulas for stiffness parameters considering $P^{ref} = 100 \text{ kN/m}^2$ as

$$E_{50}^{ref} = 60000 (D_r/100) \text{ [kN/m}^2\text{]}, \quad (3.3)$$

$$E_{oed}^{ref} = 60000 (D_r/100) \text{ [kN/m}^2\text{]}, \quad (3.4)$$

$$E_{ur}^{ref} = 180000 (D_r/100) \text{ [kN/m}^2\text{]}, \quad (3.5)$$

$$G_0^{ref} = 60000 + 68000 (D_r/100) \text{ [kN/m}^2\text{]}. \quad (3.6)$$

It is evident that the stiffness is stress-dependent. The rate of stress dependency, m , was found to correlate negatively with the density and expressed as

$$m = 0.7 - (D_r/320). \quad (3.7)$$

Poisson's ratio for unloading and reloading, ν_{ur} , is taken as 0.2. The strain level at which the small-strain shear modulus is 70% reduced, $\gamma^{0.7}$, is proposed as

$$\gamma^{0.7} = (2 - D_r/100) 10^{-4}. \quad (3.8)$$

For parameters representing the strength characteristic of the soil, the following formulas are obtained as

$$\phi' = 28 + 12.5 (D_r/100), \quad (3.9)$$

$$\psi = -2 + 12.5 (D_r/100), \quad (3.10)$$

$$R_f = 1 - (D_r/800). \quad (3.11)$$

3.2.3. Determination of Structural Parameters

The Linear Elastic model has been chosen to depict the stress-strain relationship for the structural components in parametric investigations. In the case of concrete elements, the values for Young's modulus (E), Poisson's ratio (ν), and unit weight for both the raft and the walls are set at 30 GPa, 0.2, and 25 kPa, respectively. Meanwhile, for the aluminum plates employed in laboratory experiments, the unit weight, Young's modulus, and Poisson's ratio are specified as 27 kPa, 70 GPa, and 0.3, respectively.

3.2.4. Model Size and Boundary Conditions

External boundaries were strategically positioned at a considerable distance to ensure the results achieved a level of precision deemed acceptable. Relatively close boundaries to the foundations lead to risk excluding the stresses and strains within an adequately defined area around the foundation. On the other hand, choosing large boundaries can unnecessarily extend the analysis time. As a result, it is recommended that for a BSDF or a raft with dimensions "B x B" and a foundation depth of "D", the boundary conditions be established at a distance of 5xB or 3xD (whichever is greater) in the horizontal direction and 3xD in the vertical direction.

3.2.5. Interface Elements

Interface elements are used to model the relative displacement between the soil and the structure. Using continuum elements in FEM prohibits modeling relative movement at the soil-structure interaction. The interface elements can be modeled in many methods, and Goodman et al. (1968), Langen and Vermeer (1991) and Potts and Zdravkovic (1999) have given the theory and an overview of these methods. Interface elements are essential in modeling the soil-structure interaction and accurately assessing the displacements and internal forces.

The strength reduction factor (R_{int}) is the main interface parameter. This factor relates the friction angle between the wall and the soil (δ) to the internal friction angle of the soil (ϕ). The strength reduction factor was studied by many researchers. Potyondy (1961) performed direct shear tests on the interfaces of concrete, steel, and wood with dry and wet samples of sand and clay. He proposed strength reduction factors for construction materials and the soil interfaces that ranged from 0.4 for saturated loose sand to one for saturated dense sand. Kulhawy and Peterson (1979) and Tiwari et al. (2010) also carried out direct shear tests between soil and structures to investigate strength reduction.

For considering the effect of surface roughness of the foundation models, a strength reduction factor (R_{int}) specified in the interface tab sheet of the material set in PLAXIS is to be chosen. A rigid option represented with $R_{int}=1$ is used when the interface should not have a reduced strength concerning the strength in the surrounding soil. Generally, for real soil-structure interaction, the interface is weaker and more flexible than the surrounding soil, which means the value should be less than one. PLAXIS (2019) asserts that if the necessary information is unavailable, R_{int} can be selected as 2/3.

The interface strength parameters, cohesion, c_i , friction angle, ϕ_i , and dilatancy angle ψ_i , of the interface, can be calculated as

$$c_i = R_{int} c_{soil}, \quad (3.12)$$

$$\tan \phi_i = R_{int} \tan \phi_{soil} \leq \tan \phi_{soil}, \quad (3.13)$$

$$\psi_i = 0 \text{ for } R_{int} \leq 1, \text{ otherwise } \psi_i = \psi_{soil}. \quad (3.14)$$

Therefore, in this section, three different R_{int} values of 0.6, 0.8, and one have been selected to represent the soil-structure interaction of BSDFs.

R_{int} also reduces the interface stiffness like interface strength. If the interface is elastic, slipping and gapping (overlapping) can take place. Gap displacements are correlated to effective normal stress, σ'_n at an interface and elastic interface normal stiffness, K_n and elastic slip displacements are led by shear stress, τ , and interface shear stiffness K_s . The magnitudes of the interface displacements are

$$\text{Elastic gap displacement} = \frac{\sigma'_n}{K_n} = \frac{\sigma'_n t_i}{E_{oed,i}}, \quad (3.15)$$

$$\text{Elastic slip displacement} = \frac{\tau}{K_s} = \frac{\tau t_i}{G_i}, \quad (3.16)$$

where t_i is the virtual thickness of the interface, $E_{oed,i}$ is one-dimensional compression modulus of the interface, G_i is the shear modulus of the interface, and $G_i = R_{int}^2 G$.

3.2.6. Mesh Generation

In FEM, the geometry is supposed to be divided into finite elements called meshes to perform the finite element analysis.

PLAXIS 3D generates 10-node tetrahedral elements for volume elements, 6-nodeplate elements for plates and 14-node interface elements for the soil-structure interaction in mesh the procedure.

Decreasing the mesh size increases the analysis time and, as a result, the precision of the computations. However, the mesh should be adequately fine to get the required accurate results. Moreover, very fine meshes should be avoided because they may cause excessive calculation time and numerical problems in software. Mesh coarseness can be assigned by five options:

- Very coarse,
- Coarse,
- Medium,
- Fine,
- Very fine.

It is also possible to locally refine the areas where large stress concentrations or large deformation gradients are expected with a more accurate finite element mesh when other parts of the geometry are modeled with less fine meshing.

To assess the effects of mesh coarseness, a BSDF with five different mesh fineness has been studied and the load-settlement behaviors of the different options are shown in Figure 3.7. Bearing pressures representing the values at the settlements of 50 mm and 100 mm are tabulated in Table 3.1. Therefore, in this study, structural parts were refined and mesh size was selected as “fine” for the mesh generation of the models.

Table 3.1. Coarseness effect

Mesh Size	Number of elements	Number of nodes	q^{50} (kPa)	q^{100} (kPa)
Very coarse	1588	3113	735.791	1065.32
Coarse	3170	6130	734.14	1060.35
Medium	8800	15282	730.62	1055.88
Fine	22940	37042	730.57	1055.4
Very fine	59922	92291	722.08	1040.43

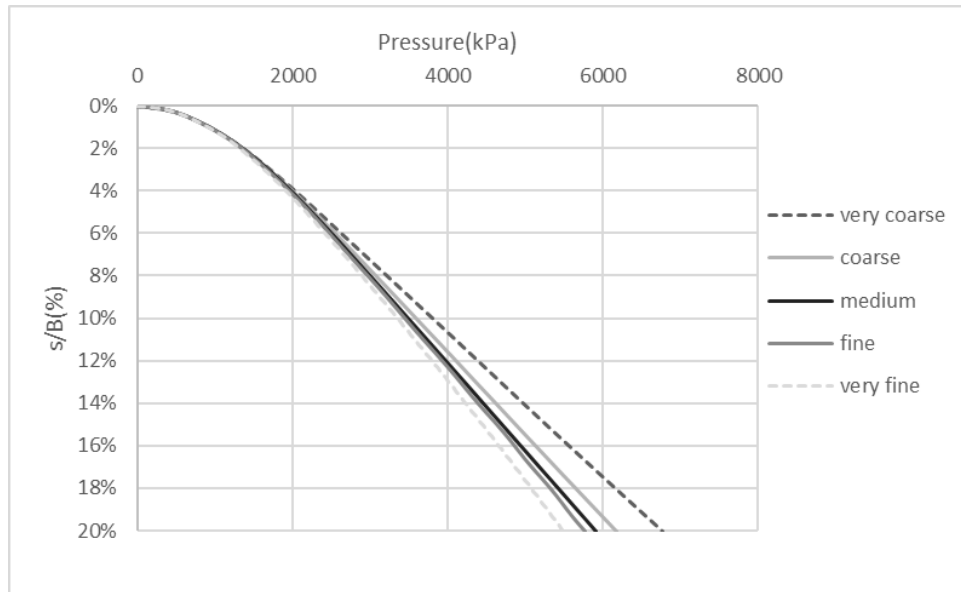


Figure 3.1. Pressure vs normalized settlement plot for different mesh coarseness.

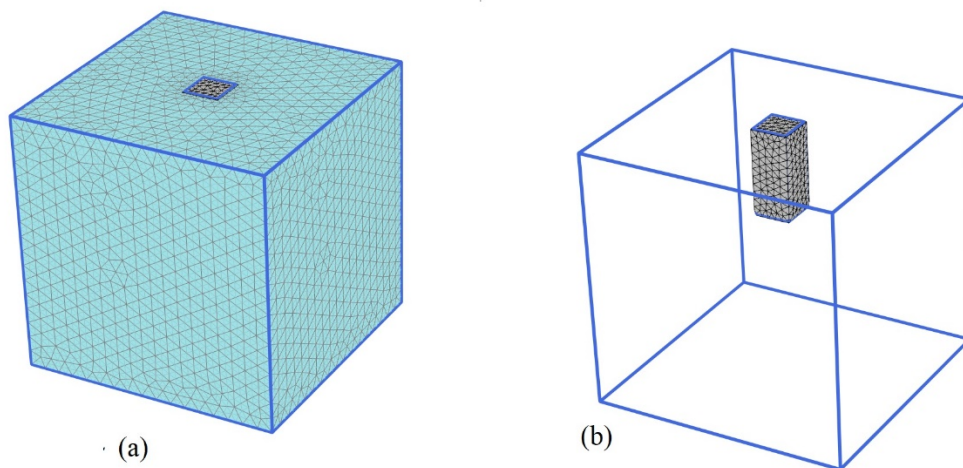


Figure 3.2. Model mesh (a) and structural mesh (b).

3.2.7. Staged Construction

The process of finite element analysis can be carried out in sequential computational stages. The process begins with the initial phase, which is characterized by the generation of initial stress conditions. In this stage, vertical stresses are harmonized with the inherent weight of the volume, while the effective horizontal stresses can be accurately computed using the lateral earth pressure coefficient, denoted as K_0 . This stage encompasses the activation of all soil volumes, while concurrently rendering all structural elements, interface elements, and loads inactive.

During the phase 1, all elements associated with the diaphragm wall plates, as well as the corresponding interface elements, are activated. Despite this, the raft, along with any associated loads or displacements, remain inactive. A noteworthy consideration in this phase is that the installation procedure of the BSDF is not simulated, and any effects from this process, if present, are assumed to have a minimal influence on the performance.

In the phase 2, all components of the diaphragm walls, the raft, and the associated interface elements are activated, while loads or displacements persist in their inactive state.

During the phase 3, a displacement control approach is utilized, where a prescribed point displacement equivalent to a settlement ratio (s/B) of 20% is conducted. Lastly, it is significant to articulate that a plastic analysis has been executed to determine the load-settlement behavior of the BSDFs. While the plastic calculation does not account for water dissipation and the variation of pore pressure with time, conducting a fully drained analysis in plastic calculations can assess long-term settlements. This method is capable of providing a sufficiently precise forecast of the ultimate settlement situation, even though the consolidation process is not explicitly analyzed.

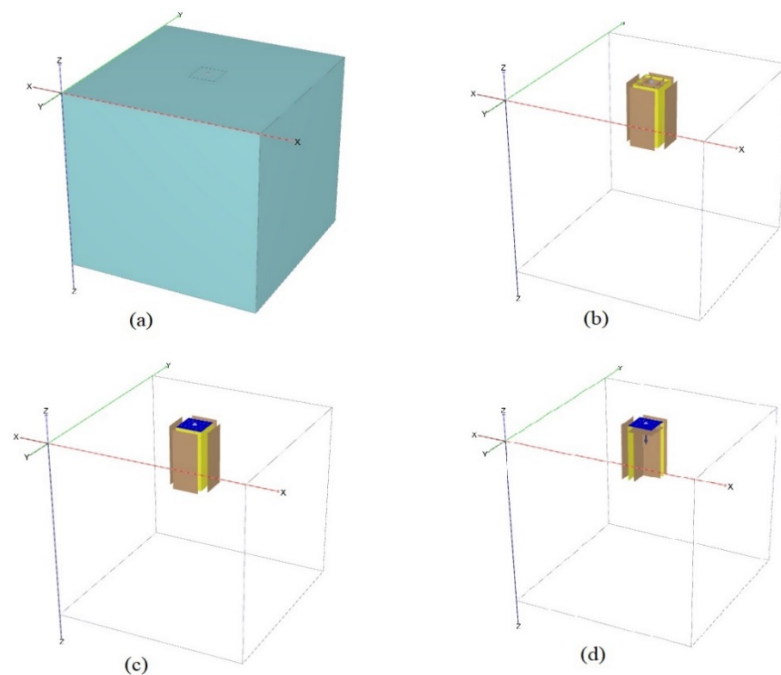


Figure 3.3. Staged construction phases.

3.3. Experimental Method

The experimental study herein aims to validate the results of numerical analysis carried out to investigate the load-settlement and monolithic behaviors. The experimental method for this validation has been submitted in this section, covering experimental setup, determination of soil properties and the experimental procedure, respectively.

3.3.1. Experimental Setup

Experimental tests were carried out inside a sturdy box with length, width and depth measurements of 100 cm x 50 cm x 65 cm, respectively. The bottom surface of the box is made entirely of steel, and it has four steel sections vertically attached to it. These sections are connected on top by another four steel sections. The box's front and rear are comprised of 10 mm thick glass, shielded by 3 mm of plexiglass. This transparent surface allows for visual monitoring of sand. On the rear glass, lines spaced 5 cm apart have been marked to verify whether the anticipated relative density is achieved while sand is being poured into the box. A depiction of the test box can be viewed in Figure 3.10.

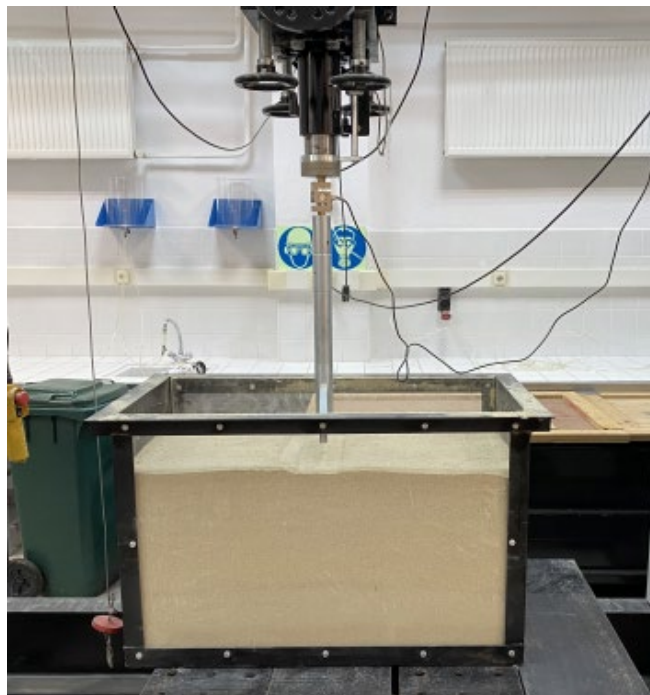


Figure 3.4. Experimental test setup.

Aluminum plates, 6 mm in thickness, were employed to simulate the walls and rafts of the BSDFs. The wall and raft components were fastened together using screws. Regarding the embedded foundations, an extra plate was installed at the bottom to prevent soil from penetrating. The dimensions of model foundations are given in Table 3.2.

Table 3.2. Model foundation dimensions.

Model Foundation	Depth (D) (cm)	Width (B) (cm)
BSDF 20x10	20	10
Embedded 20x10	20	10
BSDF 10x10	10	10
Embedded 10x10	10	10

A cylinder steel unit was installed to distribute the exerted load across the model BSDF. This steel unit was affixed to a hollow steel cylindrical piston, which was subsequently connected to both the load cell and the loading frame. In the course of the experiments, vertical footing displacements due to loadings were measured using a LVDT transducer placed on the edge of the footing with a stroke length of 100 mm. Additionally, an S-type load cell of 500 kg capacity was used to read the load values. The load cells had been calibrated by positioning recognized weights on the loading system and modifying essential coefficients to ensure precise readings. To oversee the gathered data, a computer code had been devised in the LabVIEW program. This code granted the experimenter the ability to view the metrics from the LVDT, and load cell directly on the front panel. After the testing procedure, the data were exported in the format of an Excel file, which was subsequently used for evaluation purposes.

3.3.2. Determination of Soil Properties

Çerkezköy sand was used throughout the experiments. Oner (2020) had conducted several laboratory tests to determine the physical properties of the sand. Initially, sand samples had been sieved, and particle samples retained between No 10 and 200 was also used in this study. After carrying out the sieve analysis, the grain size distribution curve

shown in Figure 3.11 has been obtained. Subsequently, D_{50} , C_u and C_c were determined by benefitting from Figure 3.11.

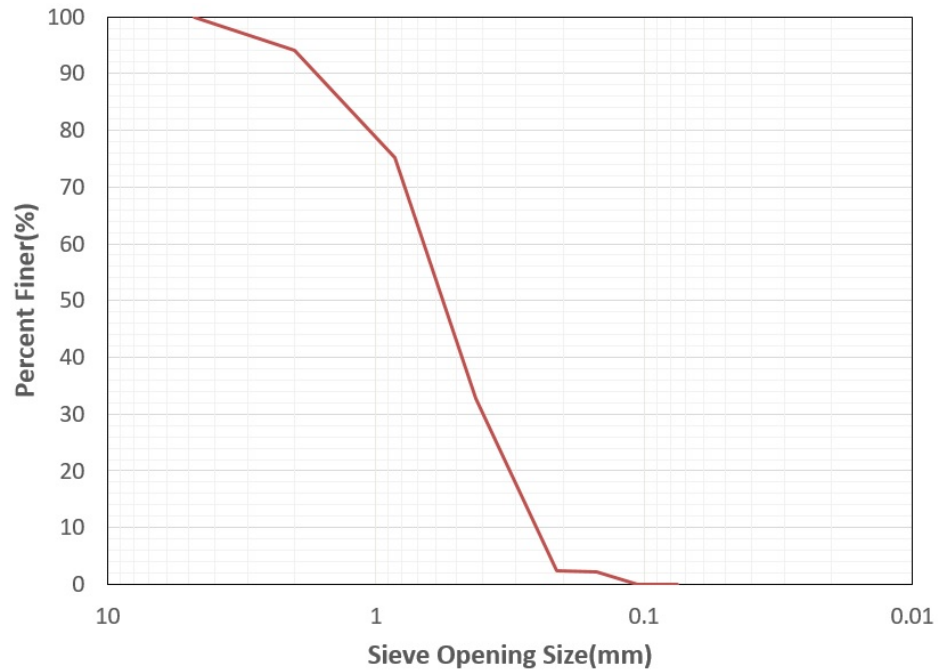


Figure 3.5. Grain size distribution curve for Çerkezköy sand.

Specific Gravity test, minimum and maximum void ratio tests were conducted for the determination of unit weigh at the target relative density. All tests were carried out according to ASTM and test results are given in Table 3.3.

Table 3.3. Physical properties of the Çerkezköy sand.

Property	Value
Classification	SP
Median Particle Size (D_{50})	0.64
Coefficient of Uniformity (C_u)	2.37
Coefficient of Curvature (C_c)	0.93
Specific Gravity (G_s)	2.62
Maximum Void Ratio (e_{max})	0.83
Minimum Void Ratio (e_{min})	0.5

As mentioned in Section 3.2, The relative density (D_r), presented as a percentage, is calculated using the formula $(e_{max}-e) / (e_{max}-e_{min})$. Here, “ e ” denotes the initial void ratio, “ e_{max} ” represents the maximum or loosest possible packing void ratio, and “ e_{min} ” signifies the minimum or densest packing void ratio. In this study, as the relative density values are already established and the maximum and minimum void ratios are provided in Table 3.3, it's possible to compute the necessary void ratio, “ e ” at the desired relative density. After determining the required void ratio, one can proceed to calculate the value of the dry unit weight as follows:

$$\gamma_d = \frac{G_s \gamma_w}{1 + e}, \quad (3.17)$$

where e is the void ratio at desired relative density, G_s is the specific gravity, γ_d and γ_w are the dry unit weight of the soil and unit weight of the water, respectively.

Following the computation of unit weight, the necessary sand weight can be calculated by multiplying the test box volume with the determined unit weight value. In experiments involving a model foundation, the volume of the foundation was subtracted from the total volume. Table 3.4 provides the mass requirements for each layer, designated as M_1 for soil mass without the foundation and M_2 for soil mass with the foundation, respectively, at specified target relative densities.

Table 3.4. Mass required at various relative densities.

Relative Density (%)	e	γ_d (kN/m ³)	ρ_d (kg/m ³)	M_1 (kg)	M_2 (kg)
35	0.71	14.99	1528.66	37.99	37.82
50	0.67	15.44	1574.11	39.12	38.94
65	0.63	15.75	1605.94	39.91	39.73

3.3.3. Experimental Procedure

The test tank was segmented into ten layers using lines spaced 5 cm apart on the plexiglass. At every layer, 10% of the necessary soil mass, based on the targeted relative density, was poured in. Following this, the sand was compacted to achieve the desired volume.

The maximum effort possible has been exerted to achieve the desired relative density, and the dry pluviation method has been used to discharge sand into the tank from a fixed distance. Vaid and Negussey (1984) investigated the relative density of pluviated sand samples. It was claimed that the achieved relative density is significantly influenced by the drop height and for a constant drop height, finer sands are likely to exhibit relatively lower densification compared to coarser sands. While the impact energies will continue to rise as drop heights increase until they reach terminal velocity, the test results suggest that beyond a specific maximum magnitude, further increases in impact energy may have limited impact on compacting the sand. Namely, there is a drop height limit beyond which insignificant compaction occurs. To ensure a consistent relative density at all layers, it is necessary to continually elevate the pouring tip as the deposit thickness increases. Based on the study conducted by Vaid and Negussey (1984), the drop height has been increased by 5 to 10 cm as the deposit thickness increases, while keeping the pouring rate constant.

The experimental process was carried out in the following manner:

- During the experiment, the target relative density of 35% was achieved by adding 10% of the necessary soil mass in each layer. Once the desired depth was attained, the box was positioned on the soil surface.
- The box was filled with sand to achieve the target relative density of 35%, and then the raft plate was affixed in place using screws.
- Following the placement of the BSDF, the soil was further filled until it reached the top level.
- The LVDT transducer was positioned at the edge of the foundation.
- The BSDF was subjected to a constant loading rate of 10 mm/min until it reached a displacement of 20 mm. Throughout the process, the LabView software recorded the load-displacement values for further evaluation.

4. PARAMETRIC STUDY

4.1. Load-settlement Behavior of BSDFs

A detailed numerical study was conducted to examine the load-settlement characteristics of BSDFs on three distinct types of sands. Initially, a representative model with dimensions of $D=20$ m, $B=8$ m, $t=0.8$ m, and $T=2$ m was simulated. Following this, 129 different model configurations were also simulated to facilitate comparison and interpretation.

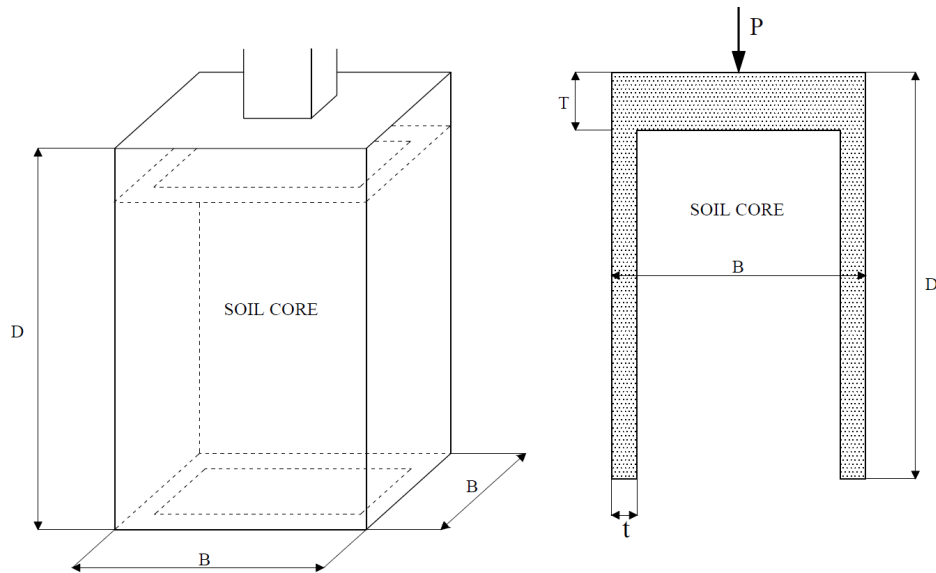


Figure 4.1. Geometry of a BSDF.

Two coefficients which are BCR and SRR have been developed to better interpret the load-settlement behavior of these foundations with different geometries, soil types and interface roughness.

BCR represents the “Bearing capacity ratio” defined as

$$BCR = q_{bsdf} / q_{shallow}, \quad (5.1)$$

where q_{bsdf} and $q_{shallow}$ are the bearing capacity of a BSDF and the bearing capacity of the representative shallow foundation, respectively.

Vesic (1977) has shown that many foundations fail when the settlement ratio (s/B) is less than 10%, but for loose to medium sand, the ultimate load (V_{ult}) could potentially reach a higher ratio of up to 25%. Consequently, the bearing capacity was computed in relation to the 10% of the s/B ratio in all instances. The detailed explanation will be given in Section 6.1.

As a second coefficient, SRR standing for the “settlement reduction ratio” is expressed as

$$SRR = s_{bsdf} / s_{shallow}, \quad (5.2)$$

where s_{bsdf} and $s_{shallow}$ are the settlement of a BSDF and the settlement of the representative shallow foundation, respectively. “ s_{bsdf} ” is measured under constant bearing pressure, which is equivalent to the bearing capacity experienced by the representative shallow foundation of the same width.

4.1.1. Numerical Modeling

In numerical simulations, HSS has been selected as the constitutive model for the soil with different relative densities. The details of this model are presented in Section 3.2.1 and Appendix 1. As for structural elements, the linear elastic model has been selected. Plate elements were used to model walls and rafts, while volume elements were employed to model piles. The water level was situated significantly below the foundation, and drained behavior was assumed. To ensure acceptable accuracy of the results, external boundaries were set at a considerable distance. For a BSDF with raft dimensions of “B x B” and depth of “D”, the boundary conditions were determined at a distance of either 5 x B or 3 x D (whichever value was larger) in the horizontal direction, and 3 x D in the vertical direction. The choice of mesh size has an impact on both computation runtime and accuracy. Smaller element sizes lead to longer analysis time but higher accuracy. Structural parts were refined and mesh size was selected as “fine” for the mesh generation of the models. More detailed scientific explanations have been provided in Section 3.2.

Using the formulas provided in Section 3.2.2, the constitutive model parameters for the sand exhibiting varying relative densities ($S_1=35\%$, $S_2=50\%$, and $S_3=65\%$) are given in Table 4.1.

Table 4.1. HSS Soil parameters for three different soils.

Parameter	Unit	S ₁	S ₂	S ₃
γ_{unsat}	[kN/m ²]	16.4	17	17.6
γ_{sat}	[kN/m ²]	19.6	19.8	20
E_{50}^{ref}	[kN/m ²]	21000	30000	39000
E_{oed}^{ref}	[kN/m ²]	21000	30000	39000
E_{ur}^{ref}	[kN/m ²]	63000	90000	117000
G_0^{ref}	[kN/m ²]	83800	94000	104200
m	[–]	0.591	0.544	0.497
$\gamma^{0.7}$	[–]	1.65E-4	1.50E-4	1.35E-4
ϕ'	[°]	32.4	34.3	36.1
ψ	[°]	2.4	4.3	6.1
R_f	[–]	0.956	0.938	0.919

For structural elements in this study, parameters used in the Linear Elastic model, which are Young's modulus E , Poison's ratio ν , and unit weight of the raft and the walls γ have been given in Table 4.2.

Table 4.2. Model parameters for structural elements.

Parameter	Unit	Value
E	[GPa]	30
ν	[–]	0.2
γ	[kN/m ²]	25

To account for the impact of surface roughness in the foundation models, a strength reduction factor (R_{int}) is determined in the interface tab sheet of the material set in PLAXIS. This factor establishes the correlation between the interface strength (wall friction and adhesion) and the soil strength (friction angle and cohesion). When the interface is not intended to exhibit reduced strength compared to the surrounding soil, a rigid option is selected with $R_{int} = 1$. Typically, in real soil-structure interaction scenarios, the interface between the foundation and the surrounding soil is comparatively weaker and more flexible

than the soil itself. Consequently, the value of R_{int} should be less than one to reflect this behavior. In this particular section, three distinct R_{int} values (0.6, 0.8, and one) have been chosen to accurately represent the soil-structure interaction. More detailed information can be found in Section 3.2.5, where additional in-depth explanations are provided.

Each analysis has been divided into sequential calculation phases which are:

- Initial Phase,
- Phase 1-Installation of the walls,
- Phase 2-Installation of the raft,
- Phase 3-Loading the BSDF.

The detailed explanation about calculation phases has been given in Section 3.2.7.

It is worth mentioning that a plastic calculation is conducted to assess the load-settlement behavior of BSDFs. Although the plastic calculation does not account for water dissipation or changes in pore pressure over time, performing a fully drained analysis in the plastic calculation can provide a long-term evaluation of settlements. This approach yields a sufficiently accurate prediction of the final state, even though the consolidation process is not explicitly analyzed.

4.1.2. Numerical Simulation Results

Figure 4.2, 4.3 and 4.4 display the numerically obtained pressure and settlement to width ratio (s/B) plots for BSDFs at aspect ratios (D/B) of 0, 0.5, 1, 1.5, 2, 2.5, 3.75 and 5 for three different sand types, namely S_1 , S_2 and S_3 which represents sands with different relative densities. In this section it should be stressed that the aspect ratios change according to changing depths since the width is constant (8m) for all cases.

Firstly, a displacement control approach was employed, imposing a prescribed point displacement equivalent to a settlement ratio (s/B) of 20%. Next, the bearing capacity was determined based on a 10% of s/B ratio. It can be clearly seen that as the depth of BSDFs

risks, so does the bearing capacity. As expected, the bearing capacity similarly increases as the relative density of the soil increases.

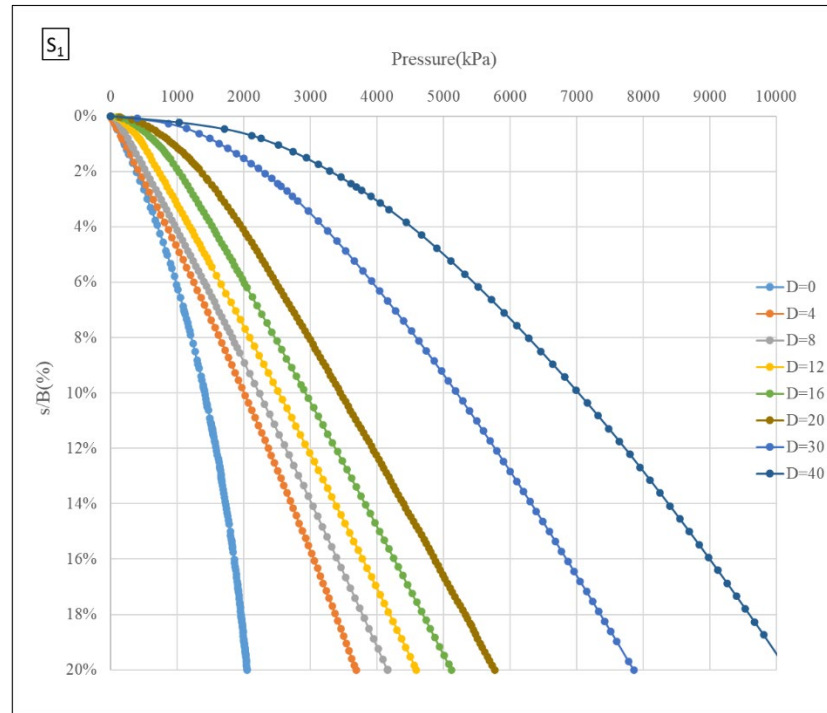


Figure 4.2. Pressure and settlement to width ratio (s/B) plots for S_1 .

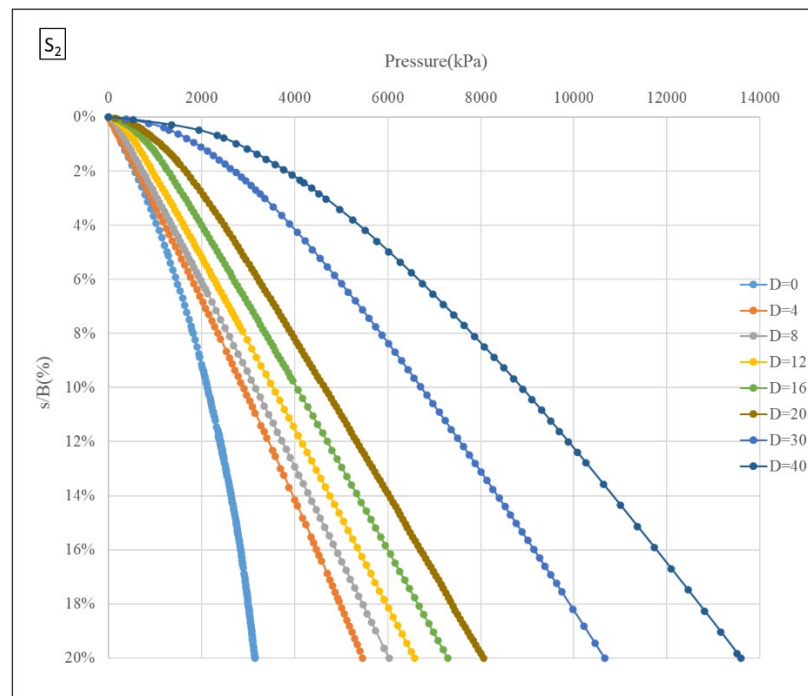


Figure 4.3. Pressure and settlement to width ratio (s/B) plots for S_2 .

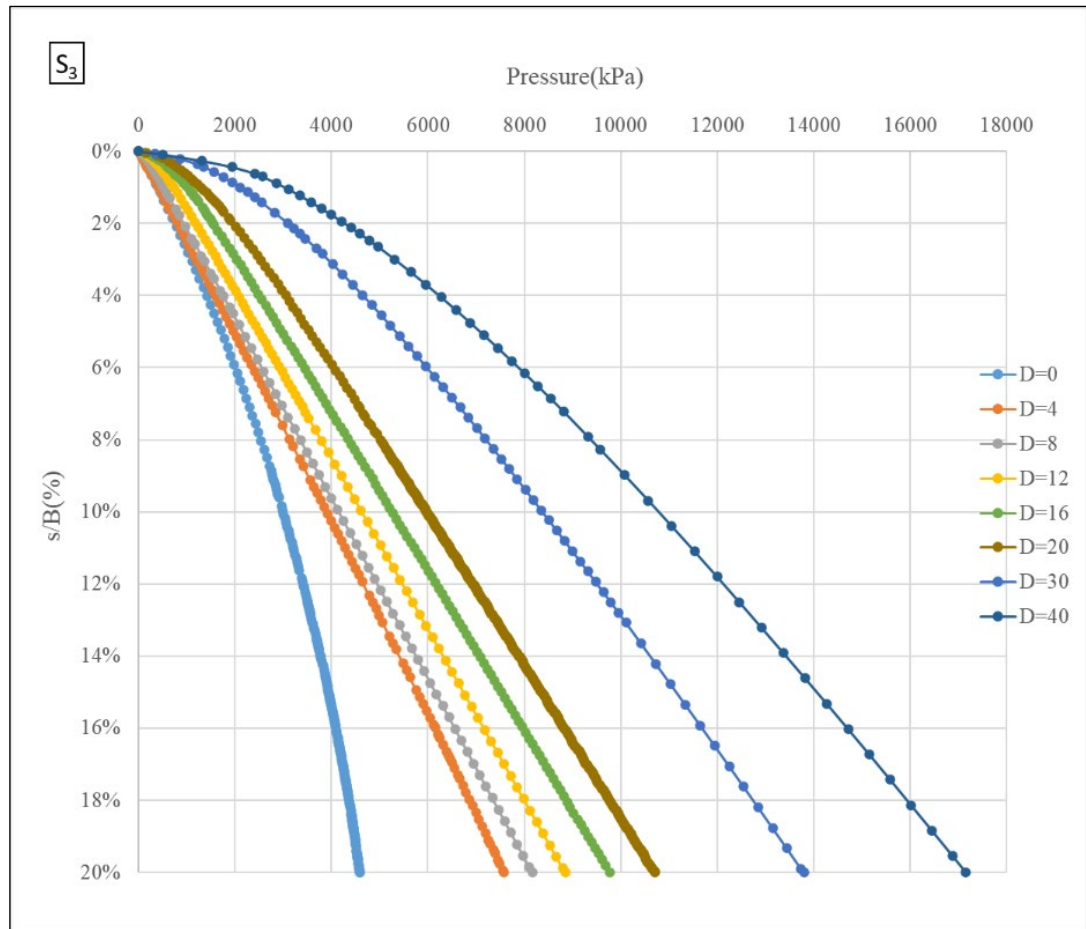


Figure 4.4. Pressure and settlement to width ratio (s/B) plots for S_3 .

The bearing capacities of foundations with different aspect ratios (D/B) have been normalized according to bearing capacity of representative shallow foundation ($D/B=0$) for three soil types. The first three tables below show the effect of depths of BSDFs on BCR for three different soils. As the depth of BSDFs increases, so does the bearing capacity and BCR. For example, the bearing capacity of a BSDF with the depth of 40 m in S_1 soil is 4.97 times higher than the shallow foundation with the same width of 8 m.

Bearing capacities of denser soils are higher for all aspect ratios as expected. However, as illustrated in Figure 4.5, the increase in BCR gets lower as the soil density increases regardless of depth and aspect ratio. Namely, the increase in BCR is higher for loose sand than medium and dense sand. Moreover, this increasing trend becomes more significant as the BSDF depth or aspect ratio increases.

Table 4.3. BCR values for S_1 considering the changing depth.

D (m)	B (m)	Aspect ratio (D/B)	S ₁ (loose sand)	
			Bearing Capacity (kPa)	BCR
0	8	0	1413.70	1.00
4	8	0.5	2010.35	1.42
8	8	1	2234.25	1.58
12	8	1.5	2527.70	1.79
16	8	2	2930.18	2.07
20	8	2.5	3455.80	2.44
30	8	3.75	5202.97	3.68
40	8	5	7030.47	4.97

Table 4.4. BCR values for S_2 considering the changing depth.

D (m)	B (m)	Aspect ratio (D/B)	S ₂ (medium dense sand)	
			Bearing Capacity (kPa)	BCR
0	8	0	2123.20	1.00
4	8	0.5	2896.53	1.36
8	8	1	3148.21	1.48
12	8	1.5	3528.92	1.66
16	8	2	4041.78	1.90
20	8	2.5	4639.82	2.19
30	8	3.75	6719.92	3.17
40	8	5	8874.27	4.18

Table 4.5 BCR values for S_3 considering the changing depth.

D (m)	B (m)	Aspect ratio (D/B)	S ₃ (dense sand)	
			Bearing Capacity (kPa)	BCR
0	8	0	2994.04	1.00
4	8	0.5	3899.47	1.30
8	8	1	4148.69	1.39
12	8	1.5	4626.48	1.55
16	8	2	5256.92	1.76
20	8	2.5	5967.22	1.99
30	8	3.75	8374.32	2.80
40	8	5	10786.83	3.60

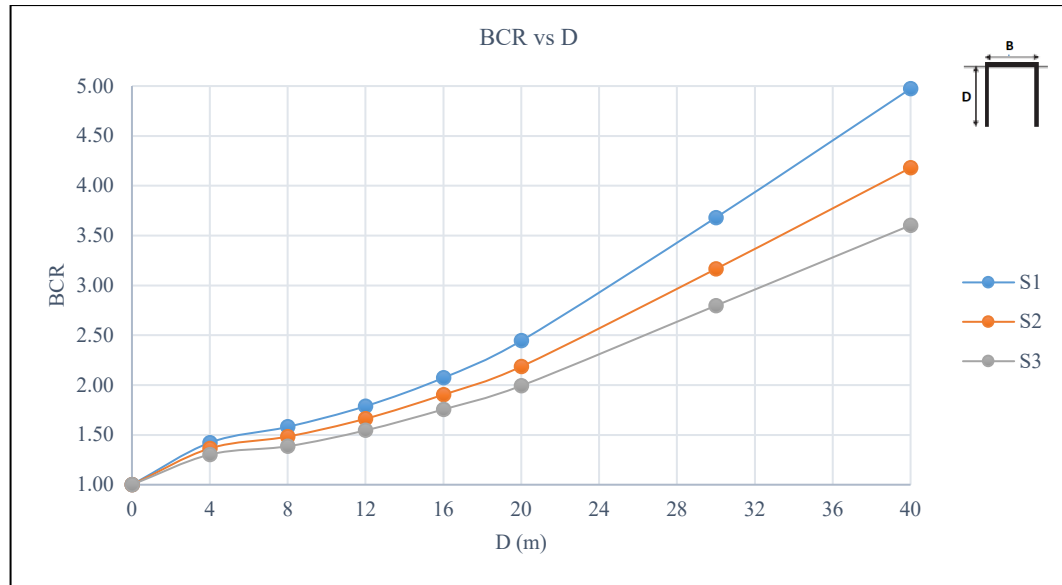


Figure 4.5. BCR vs D plots for S₁, S₂ and S₃.

Since the peak pressure was determined at an s/B of 10%, the settlement of shallow square footing corresponding to bearing capacity of 1413.7 kPa, 2123.20 kPa and 2994.04 kPa for S₁, S₂ and S₃ was found to be 80 cm. Consequently, the settlement of all BSDFs in sands S₁, S₂ and S₃ are measured corresponding to a constant bearing pressure of 1413.7 kPa, 2123.20 kPa and 2994.04 kPa as shown in Table 4.6, 4.7 and 4.8 respectively.

It can be seen from Figure 4.6 that inclusion of BSDF walls significantly reduces settlement of surface foundations for all sand types. Values of SRR that reflect the magnitude of the settlement are in reasonable agreement with BCR values, indicating that the settlement reduction inclination is higher for looser sands.

Table 4.6. SRR values for S₁ considering the changing depth.

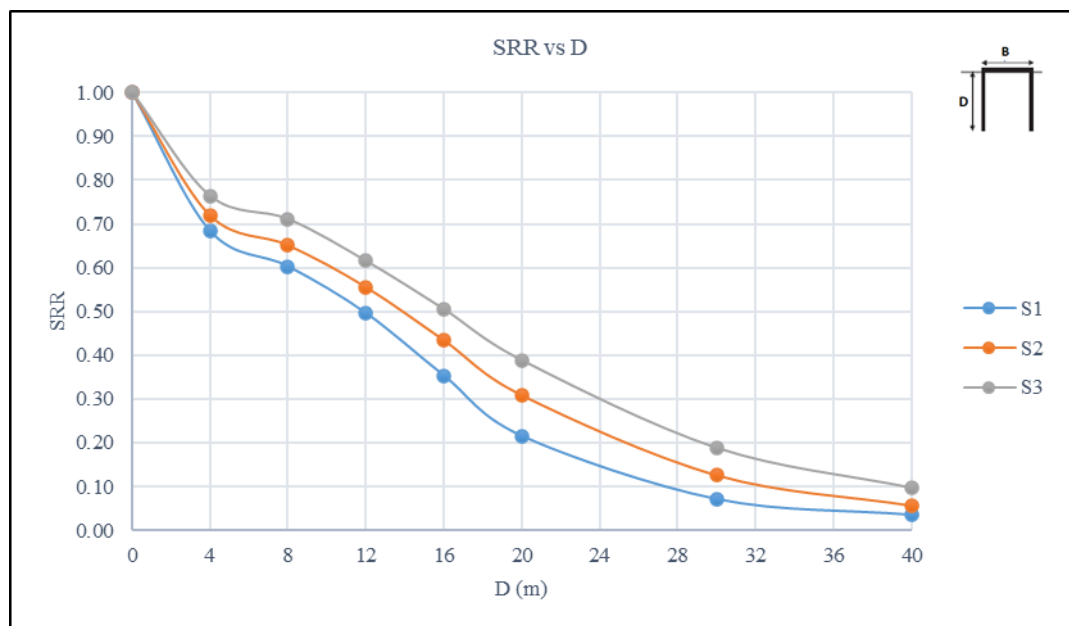
D (m)	B (m)	Aspect ratio (D/B)	S ₁ (loose sand)	
			Settlement (cm)	SRR
0	8	0	80.00	1.00
4	8	0.5	54.73	0.68
8	8	1	48.27	0.60
12	8	1.5	39.72	0.50
16	8	2	28.30	0.35
20	8	2.5	17.32	0.22
30	8	3.75	5.79	0.07
40	8	5	2.91	0.04

Table 4.7. SRR values for S_2 considering the changing depth.

D (m)	B (m)	Aspect ratio (D/B)	S_2 (medium dense sand)	
			Settlement (cm)	SRR
0	8	0	80.00	1.00
4	8	0.5	57.55	0.72
8	8	1	52.12	0.65
12	8	1.5	44.47	0.56
16	8	2	34.76	0.43
20	8	2.5	24.75	0.31
30	8	3.75	10.16	0.13
40	8	5	4.62	0.06

Table 4.8. SRR values for S_3 considering the changing depth.

D (m)	B (m)	Aspect ratio (D/B)	S_3 (dense sand)	
			Settlement (cm)	SRR
0	8	0	80.00	1.00
4	8	0.5	61.13	0.76
8	8	1	56.89	0.71
12	8	1.5	49.24	0.62
16	8	2	40.42	0.51
20	8	2.5	31.11	0.39
30	8	3.75	15.10	0.19
40	8	5	7.81	0.10

Figure 4.6. SRR vs D plots for S_1 , S_2 and S_3 .

BSDFs with different widths but constant depths have been simulated to observe the load-settlement behavior with the changing width. It is important to note that the bearing capacities have been calculated for settlements equal to 10% of the widths for BSDFs with the same depths but varying widths. In addition to submitted models, seven additional models shown in Table 4.12 representing shallow foundations with different widths have been modelled in order to normalize and find SRR results values for S_1 , S_2 and S_3 .

Figure 4.7 and 4.8 demonstrate a decrease in BCR values and an increase in SRR values as the width of the BSDFs increases. Moreover, the increase in bearing capacity and the inclination towards settlement reduction are more significant for loose sand compared to medium and dense sand considering the changing widths. Additionally, this trend becomes less significant after the width of 20 m, and relative density has less effect on BCR. Additionally, the decrease in BCR values and the increase in SRR values with the increasing width reaches a maximum and minimum point at the width of 20 m, respectively, and shows steady changes afterwards.

Table 4.9. BCR and SRR values for S_1 considering the changing width.

S_1 (loose sand)						
D (m)	B (m)	Aspect ratio (D/B)	Bearing Capacity (kPa)	BCR	Settlement (cm)	SRR
20	4	5.00	4077.91	4.42	1.51	0.04
20	8	2.50	3455.80	2.44	17.32	0.22
20	12	1.67	3341.22	1.88	51.03	0.43
20	16	1.25	3332.41	1.55	93.15	0.58
20	20	1.00	3342.63	1.35	140.96	0.70
20	30	0.67	3243.46	1.16	254.46	0.85
20	40	0.50	2841.88	1.13	350.63	0.88

Table 4.10. BCR and SRR values for S_2 considering the changing width.

S_2 (medium dense sand)						
D (m)	B (m)	Aspect ratio (D/B)	Bearing Capacity (kPa)	BCR	Settlement (cm)	SRR
20	4	5.00	5456.01	3.85	3.07	0.08
20	8	2.50	4639.82	2.19	24.75	0.31
20	12	1.67	4572.27	1.74	60.73	0.51
20	16	1.25	4508.67	1.45	104.37	0.65
20	20	1.00	4456.74	1.28	152.08	0.76
20	30	0.67	4107.92	1.13	261.91	0.87
20	40	0.50	3435.96	1.12	354.89	0.89

Table 4.11. BCR and SRR values for S_3 considering the changing width.

S_3 (dense sand)						
D (m)	B (m)	Aspect ratio (D/B)	Bearing Capacity (kPa)	BCR	Settlement (cm)	SRR
20	4	5.00	7039.45	3.50	4.81	0.12
20	8	2.50	5967.22	1.99	31.11	0.39
20	12	1.67	5897.06	1.61	69.05	0.58
20	16	1.25	5737.37	1.36	114.27	0.71
20	20	1.00	5582.58	1.21	161.96	0.81
20	30	0.67	4890.95	1.12	267.79	0.89
20	40	0.50	3962.96	1.10	363.33	0.91

Table 4.12. Bearing capacity for shallow foundations.

D (m)	B (m)	Bearing Capacity (kPa)		
		S_1	S_2	S_3
0	4	923.34	1417.73	2011.17
0	8	1413.70	2123.20	2994.04
0	12	1772.68	2622.15	3664.24
0	16	2143.46	3112.46	4230.71
0	20	2483.06	3489.68	4595.19
0	30	2797.10	3621.96	4383.60
0	40	2512.02	3063.26	3604.81

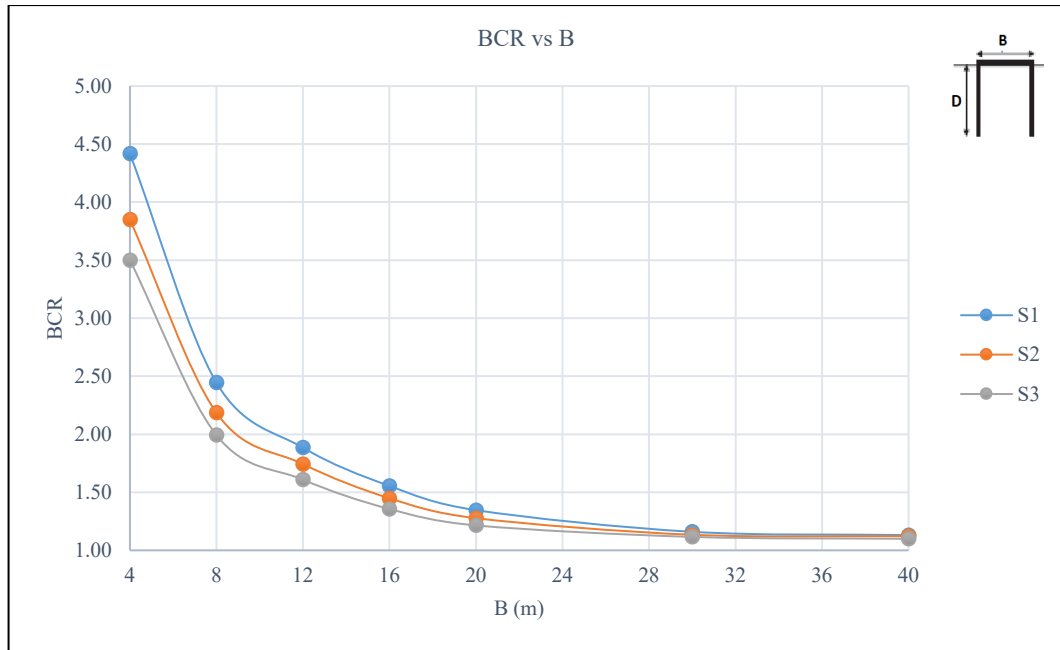


Figure 4.7. BCR vs B plots for S_1 , S_2 and S_3 .

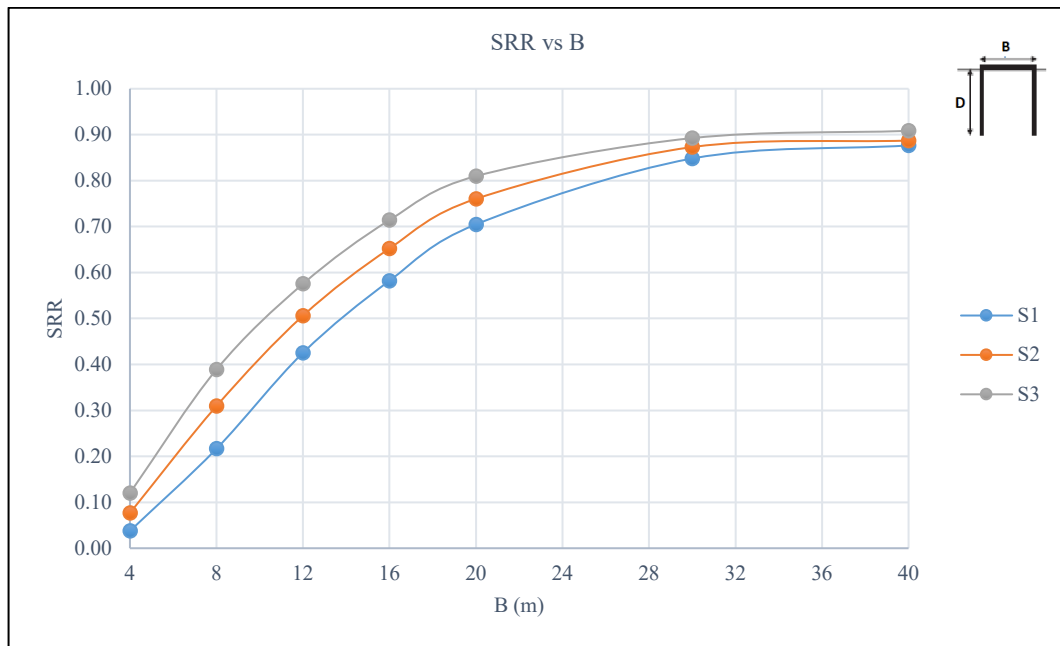


Figure 4.8. SRR vs B plots for S_1 , S_2 and S_3 .

The effect of R_{int} on BCR and SRR has been investigated by conducting several simulations with different R_{int} values which are selected as 0.6, 0.8 and one. As R_{int} increases, both bearing capacity and BCR increase. Moreover, the results indicate that the enhancement in bearing capacity values correlates with the increasing surface roughness of the foundation

models. However, settlements and SRR values decrease with the increasing R_{int} . BCR and SRR values remain similar for different R_{int} values until the depth of four, however, they tend to disperse afterwards.

Table 4.13. BCR vs D values for $R_{int}=0.6, 0.8$ and 1 .

D (m)	B (m)	Aspect ratio (D/B)	$R_{int}=0.6$		$R_{int}=0.8$		$R_{int}=1$	
			Bearing Capacity (kPa)	BCR	Bearing Capacity (kPa)	BCR	Bearing Capacity (kPa)	BCR
0	8	0	1393.28	1.00	1413.70	1.00	1449.20	1.00
4	8	0.5	1962.17	1.41	2010.35	1.42	2107.94	1.45
8	8	1	2075.63	1.49	2234.25	1.58	2582.31	1.78
12	8	1.5	2197.52	1.58	2527.70	1.79	3173.12	2.19
16	8	2	2390.10	1.72	2930.18	2.07	3835.23	2.65
20	8	2.5	2654.93	1.91	3455.80	2.44	4570.91	3.15
30	8	3.75	3628.10	2.60	5202.97	3.68	6684.73	4.61
40	8	5	4760.08	3.42	7030.47	4.97	9062.83	6.25

Table 4.14. SRR vs D values for $R_{int}=0.6, 0.8$ and 1 .

D (m)	B (m)	Aspect ratio (D/B)	$R_{int}=0.6$		$R_{int}=0.8$		$R_{int}=1$	
			Settlement (cm)	SRR	Settlement (cm)	SRR	Settlement (cm)	SRR
0	8	0	80.00	1.00	80.00	1.00	80.00	1.00
4	8	0.5	55.50	0.69	54.73	0.68	52.51	0.66
8	8	1	52.93	0.66	48.27	0.60	39.90	0.50
12	8	1.5	48.82	0.61	39.72	0.50	29.09	0.36
16	8	2	42.38	0.53	28.30	0.35	20.28	0.25
20	8	2.5	34.00	0.43	17.32	0.22	14.08	0.18
30	8	3.75	14.20	0.18	5.79	0.07	4.99	0.06
40	8	5	4.27	0.05	2.91	0.04	2.64	0.03

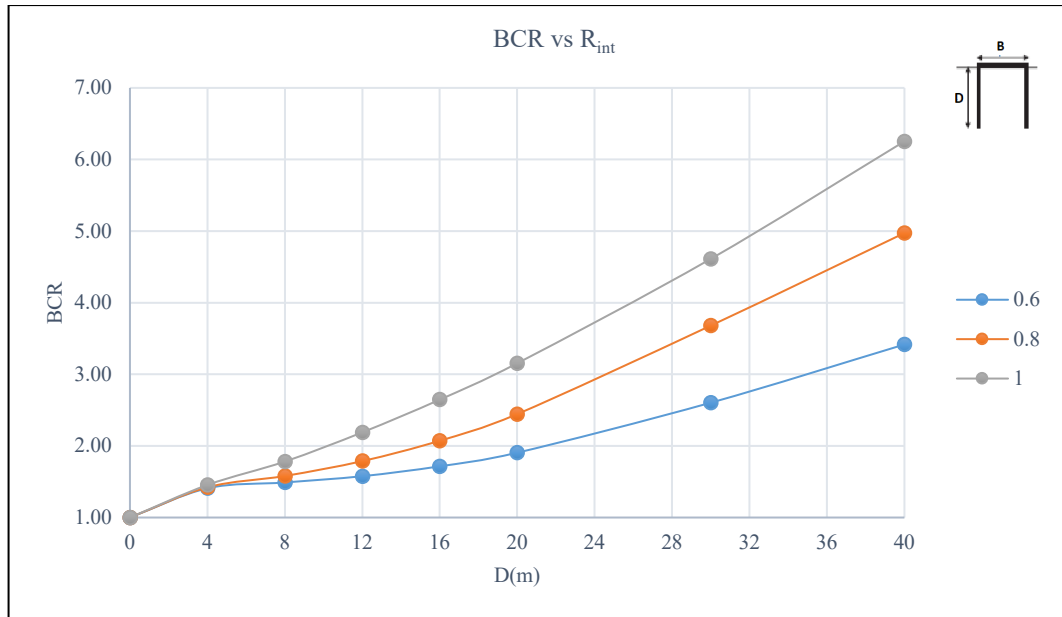


Figure 4.9. BCR vs D plots for $R_{int}=0.6, 0.8$ and 1.

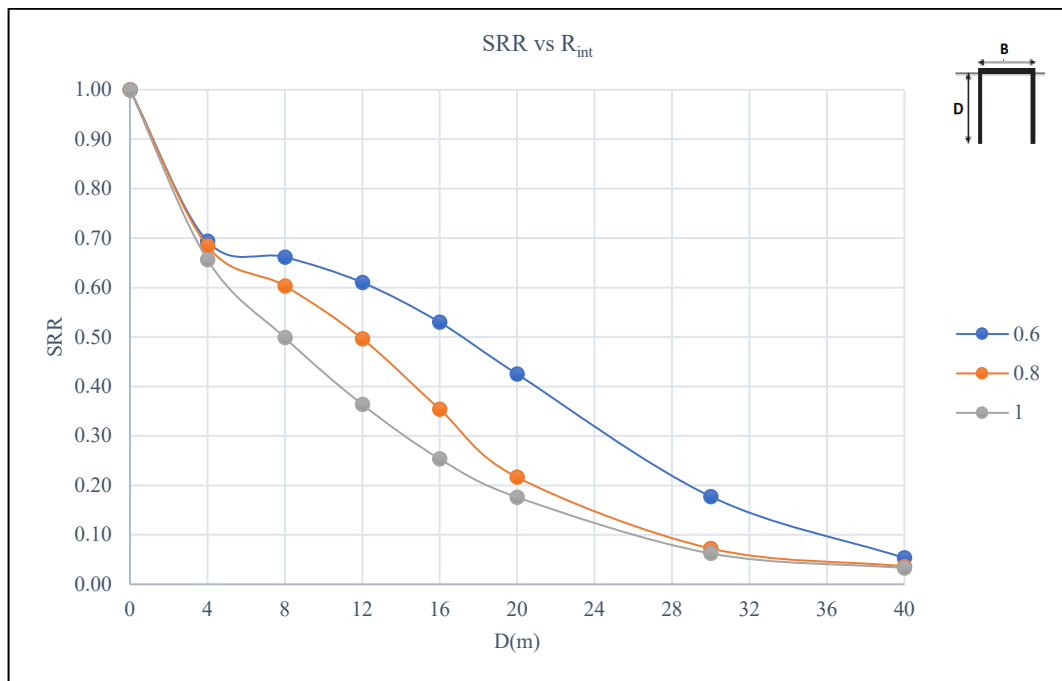


Figure 4.10. SRR vs D plots for $R_{int}=0.6, 0.8$ and 1.

Based on the data from this study, BCR values show a tendency to increase, while SRR values show a tendency to decrease as the aspect ratio increases. However, it is essential to recognize that geometries with the same aspect ratios might exhibit varying BCR and SRR values.

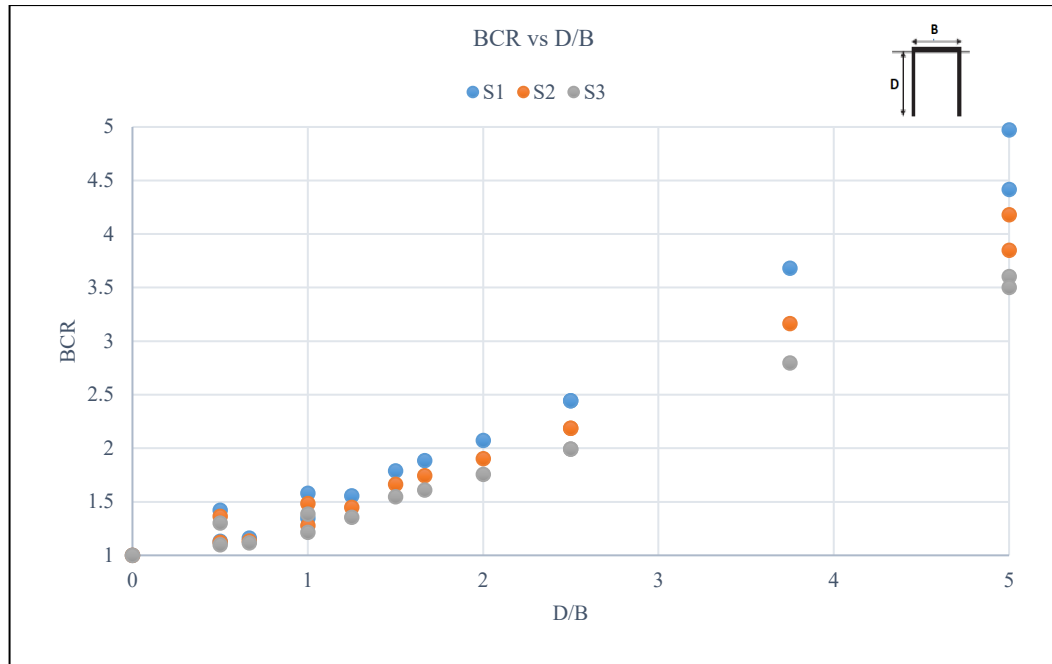


Figure 4.11. BCR vs D/B plots for S_1 , S_2 and S_3 .

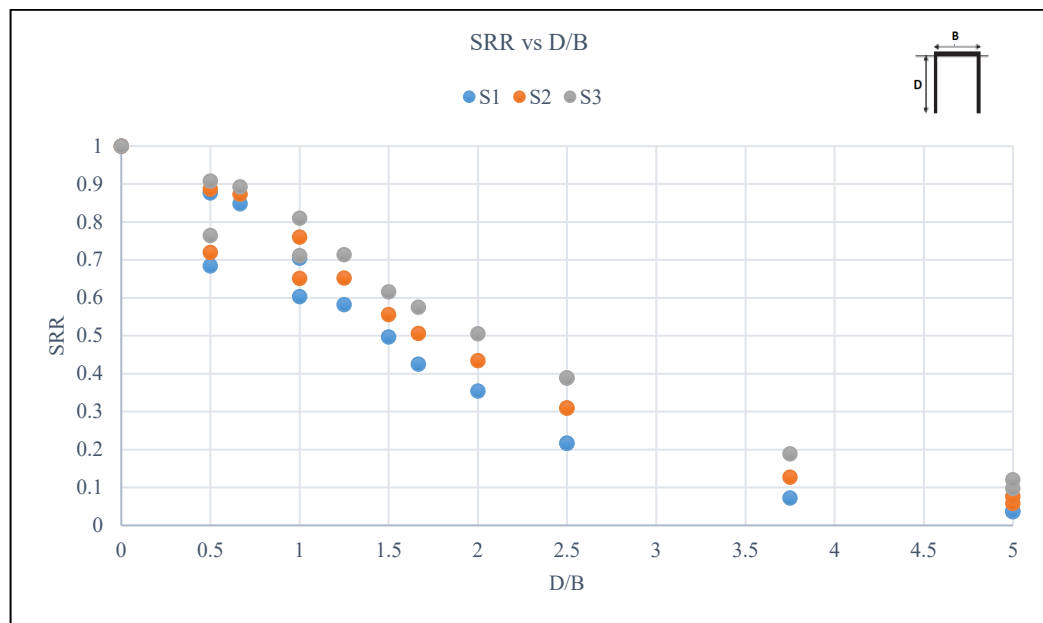


Figure 4.12. SRR vs D/B plots for S_1 , S_2 and S_3 .

Until now, BCR values were computed by assessing the bearing capacity at a settlement equivalent to 10% of the B . However, in order to account for various load scenarios, BCR values have also been determined by evaluating bearing pressures at settlements corresponding to 1.25%, 2.5%, and 5% of the B .

Table 4.15 and Figure 4.13 depict the effect of different load levels on BCR values for the changing depths. For settlements with magnitudes equal to 1.25%, 2.5%, 5% and 10% of B, BCR values increase with the growing depth of BSDFs. However, the BCR values which initially exhibit higher values at higher load levels up to a certain depth, begins to decrease as the load level increases over time.

Table 4.15. BCR vs D values for different load levels.

D (m)	B (m)	D/B	BCR			
			s=0.0125B	s=0.025B	s=0.05B	s=0.1B
0	8	0	1.00	1.00	1.00	1.00
4	8	0.5	1.07	1.13	1.23	1.42
8	8	1	1.44	1.37	1.40	1.58
12	8	1.5	2.17	1.78	1.67	1.79
16	8	2	3.18	2.45	2.08	2.07
20	8	2.5	4.22	3.24	2.62	2.44
30	8	3.75	7.26	5.41	4.20	3.68
40	8	5	10.83	7.76	5.87	4.97

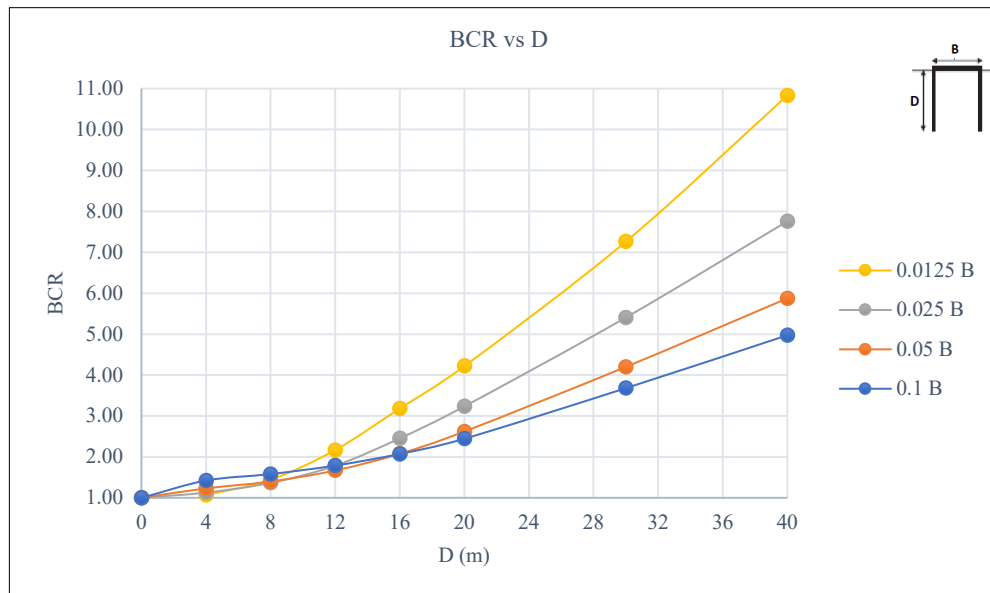


Figure 4.13. BCR vs D plots for different load levels.

Table 4.16 and Figure 4.14 illustrate the effect of different load levels on BCR values for the changing widths. For all load levels, there is a rapid decrease in BCR values as width increases and BCR values tend to decrease as load levels increase.

Table 4.16. BCR vs B values for different load levels.

D (m)	B (m)	D/B	BCR			
			s=0.0125B	s=0.025B	s=0.05B	s=0.1B
20	4	5.00	6.98	5.57	4.68	4.42
20	8	2.50	4.22	3.24	2.62	2.44
20	12	1.67	2.64	2.09	1.83	1.88
20	16	1.25	1.97	1.63	1.50	1.55
20	20	1.00	1.61	1.40	1.31	1.35
20	30	0.67	1.27	1.19	1.15	1.16
20	40	0.50	1.13	1.11	1.11	1.13

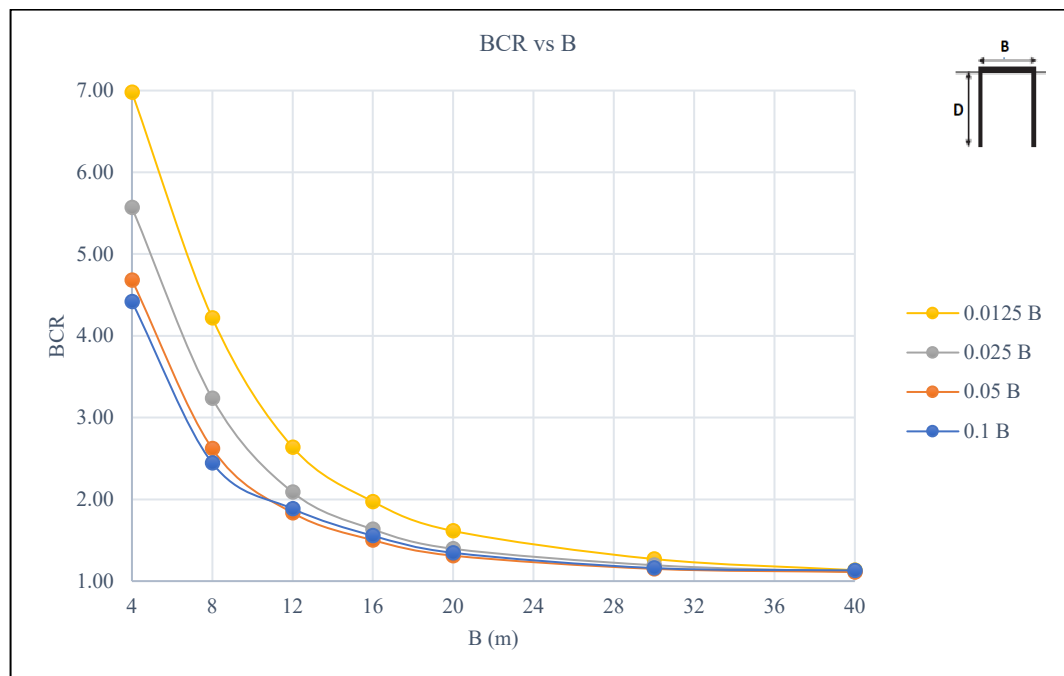


Figure 4.14. BCR vs B plots for different load levels.

4.2. Monolithic Behavior of BSDFs

The primary goal of a BSDF is to form a soil volume within the walls and establish a rigid core that functions as an integrated load transfer element. However, the inclusion of the soil core introduces complexities in the behavior of BSDFs. To address this challenge, two approaches have been proposed to examine the degree to which a BSDF functions as a compound body.

The first approach, known as the monolithic approach, considers the complete composite effect between the soil core and the structural elements. It takes into account the skin friction along the exterior surface of the box and the bearing capacity of an equivalent shallow foundation at the bottom to determine the overall bearing capacity of the BSDF. The second approach, called the single element approach, accounts for skin friction along both the exterior and interior faces of the walls. Additionally, it separately calculates the tip resistance of the walls and the bearing capacity of the raft. As a result, the actual behavior of the BSDFs lies between these two approaches.

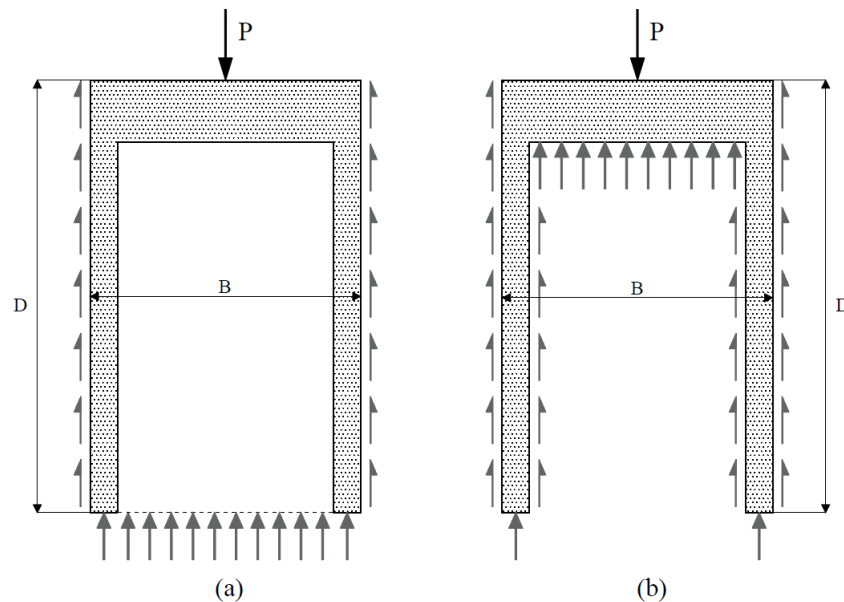


Figure 4.15. Monolithic approach (a) and single element approach (b).

Drawing inspiration from the two approaches, additional cases involving embedded foundations have been modelled to resemble typical BSDFs. The key difference is that the bottoms of these embedded foundations are closed, preventing soil penetration. In this section, a comparison will be made between these embedded foundations, representing monolithic blocks, and BSDFs.

It is evident that the embedded foundations exhibit higher bearing pressure and less settlement than BSDFs. However, as the aspect ratio increases, the disparities in bearing capacities and settlements between the two types of foundations change.

To better interpret the monolithic behaviour of BSDFs with varying geometries, soil types, and interface roughness, two coefficients have been introduced: MBR and MSR.

MBR represents the “monolithic bearing ratio” defined as

$$\text{MBR} = q_{\text{bsdf}} / q_{\text{embedded}}, \quad (5.3)$$

where q_{bsdf} and q_{embedded} are the bearing capacity of a BSDF and the bearing capacity of the representative embedded foundation, respectively.

As a second coefficient, MSR standing for the “monolithic settlement ratio” is expressed as

$$\text{MSR} = s_{\text{bsdf}} / s_{\text{embedded}}, \quad (5.4)$$

where s_{bsdf} and s_{embedded} are the settlement of a BSDF and the settlement of the representative embedded foundation, respectively. “ s_{bsdf} ” and “ s_{embedded} ” are measured under constant bearing pressure, which is equivalent to the bearing capacity experienced by the representative BSDF of the same depth or the width.

The MBR and MSR values serve as indicators to determine the degree to which a BSDF behaves like a compound body. The closer the MBR and MSR values get to one, the more the BSDF behaves like a monolithic compound body.

4.2.1. Numerical Modeling

The numerical modelling assumptions and parameters used in numerical analyses for monolithic behavior assessments are shown in Section 4.1.1 and Section 3.2.

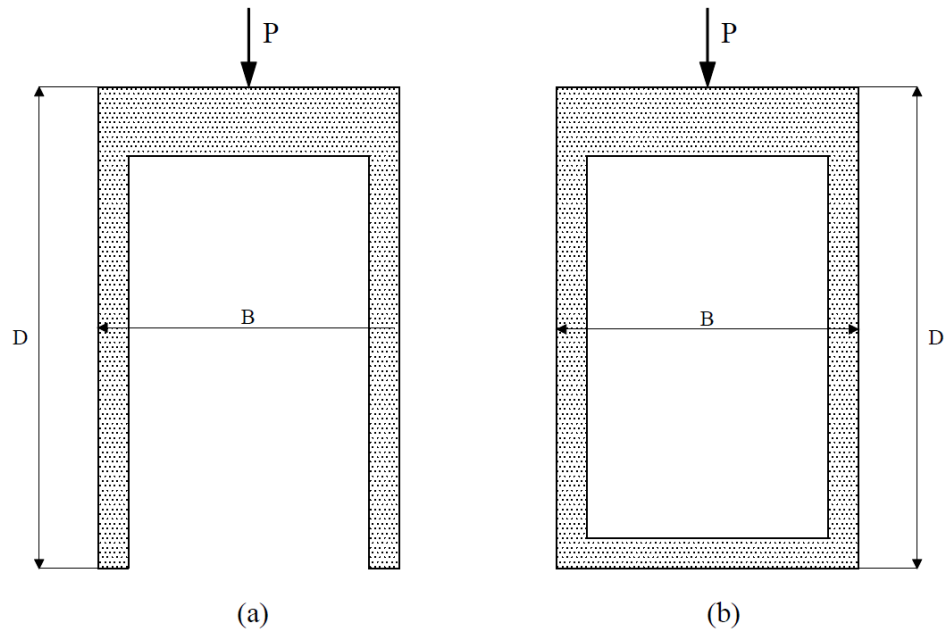


Figure 4.16. The geometry of a typical BSDF (a) and an embedded foundation (b).

4.2.2. Numerical Simulation Results

Similar to Section 4.1, the bearing capacity was determined based on a 10% of s/B ratio throughout the monolithic behaviour assessments in this section. Figure 4.17 illustrates that embedded foundations have higher bearing pressure and less settlement, compared to BSDFs. However, the differences of bearing capacities and settlements between two types of foundations vary as aspect ratio rises. MBR and MSR are indicators of this variation based on changing geometry, soil roughness and soil type.

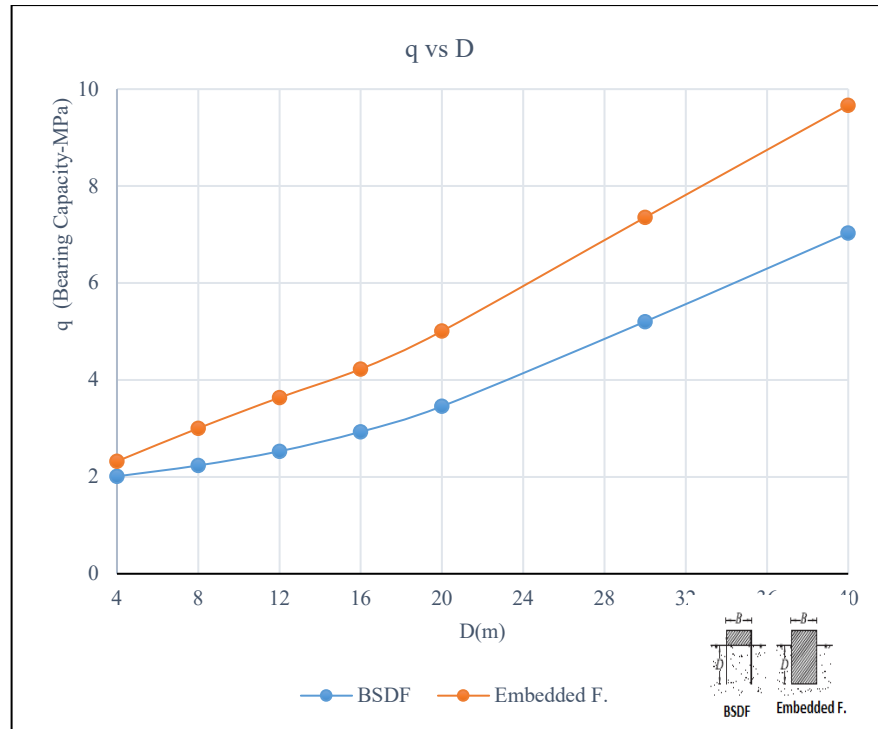


Figure 4.17. q vs D plots for BSDFs and embedded counterparts with $R_{int}=0.8$.

Table 4.15, 4.16 and 4.17 present the MBR values for the changing depths for S_1 , S_2 and S_3 . For instance, Table 4.15 and Figure 4.16 show that there is a rapid decrease in MBR values for S_1 initially, after the depth of 20 m reaching a plateau of 0.69 and rising to 0.73 again. Figure 4.16 illustrates that the same trend is observed for the other sand types. As for the impact of the soil density, BSDFs lying in denser soil show slightly higher MBR values compared to these in loose soil.

Table 4.17. MBR values for S_1 considering the changing depths.

S_1 (loose sand)					
D (m)	B (m)	Aspect ratio (D/B)	BSDF	Embedded Foundation	MBR
			Bearing Capacity (kPa)	Bearing Capacity (kPa)	
4	8	0.5	2010.35	2320.65	0.87
8	8	1	2234.25	3001.33	0.74
12	8	1.5	2527.70	3634.30	0.70
16	8	2	2930.18	4225.66	0.69
20	8	2.5	3455.80	5006.35	0.69
30	8	3.75	5202.97	7353.06	0.71
40	8	5	7030.47	9668.99	0.73

Table 4.18. MBR values for S₂ considering the changing depths.

S ₂ (medium dense sand)					
D (m)	B (m)	Aspect ratio (D/B)	BSDF	Embedded Foundation	MBR
			Bearing Capacity (kPa)	Bearing Capacity (kPa)	
4	8	0.5	2896.53	3315.78	0.87
8	8	1	3148.21	4165.19	0.76
12	8	1.5	3528.92	4966.61	0.71
16	8	2	4041.78	5729.13	0.71
20	8	2.5	4639.82	6723.39	0.69
30	8	3.75	6719.92	9477.62	0.71
40	8	5	8874.27	12193.08	0.73

Table 4.19. MBR values for S₃ considering the changing depths.

S ₃ (dense sand)					
D (m)	B (m)	Aspect ratio (D/B)	BSDF	Embedded Foundation	MBR
			Bearing Capacity (kPa)	Bearing Capacity (kPa)	
4	8	0.5	3899.47	4483.35	0.87
8	8	1	4148.69	5524.62	0.75
12	8	1.5	4626.48	6512.29	0.71
16	8	2	5256.92	7473.50	0.70
20	8	2.5	5967.22	8667.09	0.69
30	8	3.75	8374.32	11737.95	0.71
40	8	5	10786.83	14834.14	0.73

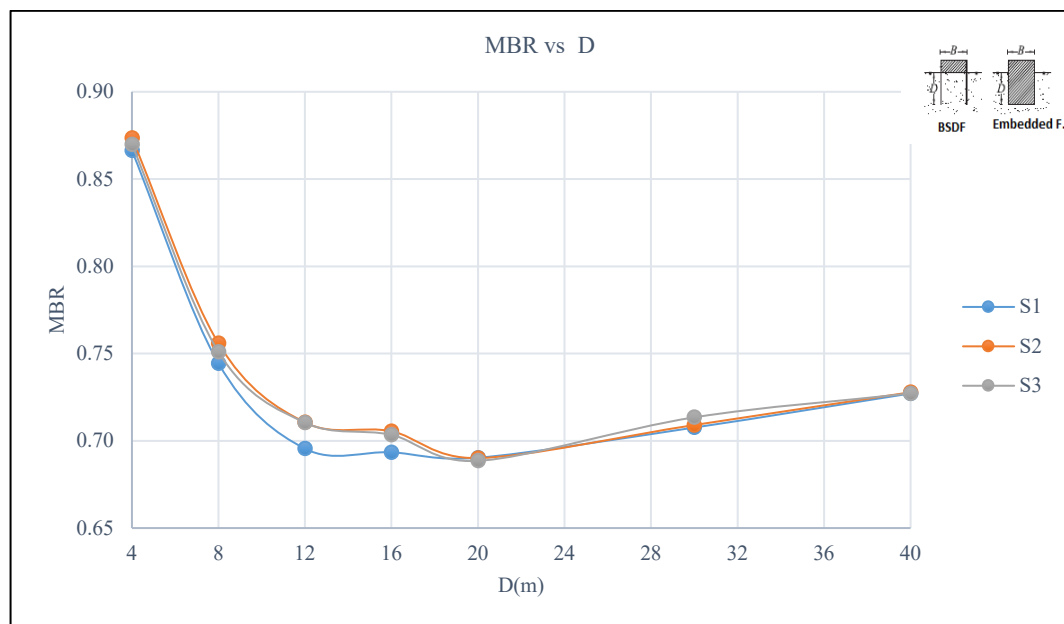
Figure 4.18. MBR vs D plots for S₁, S₂ and S₃.

Table 4.18, 4.19 and 4.20 show the MSR values for the changing depths and relative density. For instance, Table 4.18 shows that there is a substantial increase in MSR values for S_1 initially, reaching a value of 1.78 at the depth of 16 m and continue increasing gradually. Considering the impact of the soil density, BSDFs lying in denser soil show lower MSR values compared to these in loose soil.

Table 4.20. MSR values for S_1 considering the changing depths.

S_1 (loose sand)					
D (m)	B (m)	Aspect ratio (D/B)	BSDF	Embedded Foundation	MSR
			Settlement (cm)	Settlement (cm)	
4	8	0.5	80.00	67.07	1.19
8	8	1	80.00	54.15	1.48
12	8	1.5	80.00	47.57	1.68
16	8	2	80.00	44.93	1.78
20	8	2.5	80.00	43.71	1.83
30	8	3.75	80.00	42.27	1.89
40	8	5	80.00	41.40	1.93

Table 4.21. MSR values for S_2 considering the changing depths.

S_2 (medium dense sand)					
D (m)	B (m)	Aspect ratio (D/B)	BSDF	Embedded Foundation	MSR
			Settlement (cm)	Settlement (cm)	
4	8	0.5	80.00	68.27	1.17
8	8	1	80.00	56.56	1.41
12	8	1.5	80.00	50.80	1.57
16	8	2	80.00	48.11	1.66
20	8	2.5	80.00	46.49	1.72
30	8	3.75	80.00	45.71	1.75
40	8	5	80.00	45.45	1.76

Table 4.22. MSR values for S_3 considering the changing depths.

S_3 (dense sand)					
D (m)	B (m)	Aspect ratio (D/B)	BSDF	Embedded Foundation	MSR
			Settlement (cm)	Settlement (cm)	
4	8	0.5	80.00	68.50	1.17
8	8	1	80.00	57.44	1.39
12	8	1.5	80.00	52.37	1.53
16	8	2	80.00	50.10	1.60
20	8	2.5	80.00	48.46	1.65
30	8	3.75	80.00	48.35	1.65
40	8	5	80.00	48.05	1.66

Generally speaking, both MBR and MSR values, as depth increases, initially move away from one considerably, but after a certain point, this trend changes, and these values start to level off.

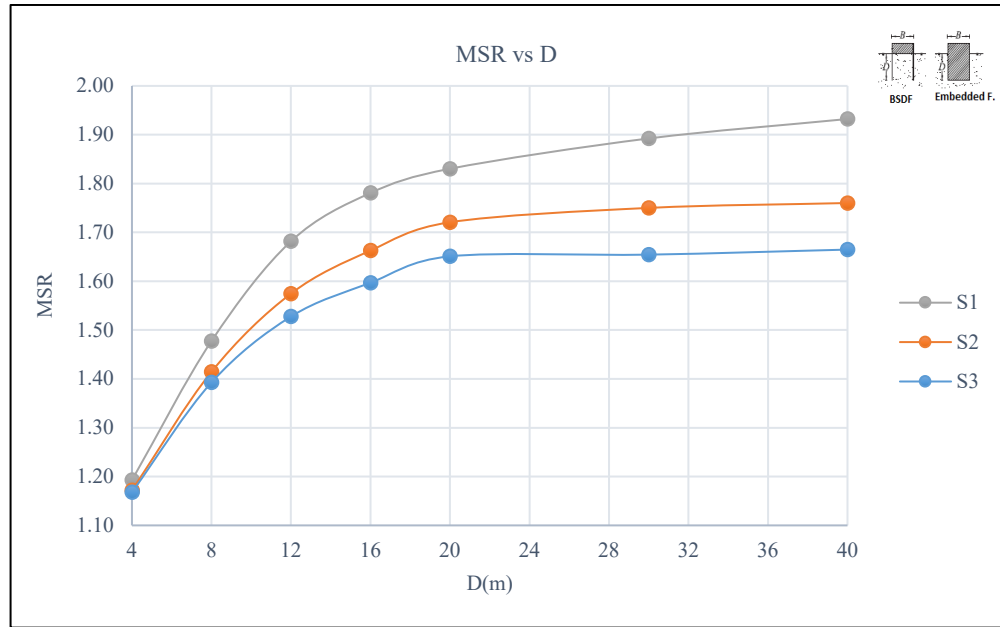


Figure 4.19. MSR vs D plots for S₁, S₂ and S₃.

Simulations have been conducted on BSDFs of constant depths but varying widths to observe how MBR changes with the changing width. Similar to Section 4.1, bearing capacities have been computed for settlements equivalent to 10% of the widths for BSDFs sharing the same depths but different widths. Table 4.21, 4.22 and 4.23 present the MBR values for the changing width and relative density. As an example, Table 4.21 shows that there is a modest decrease in MBR values for S₁ initially, reaching a minimum of 0.65 and later rising to 0.91.

Figure 4.20 displays that there is a slight increase in MBR values initially as width increases, but reaching a minimum value of 0.65 after the depth of 12 m, and afterwards MSR values begin to increase. Considering the impact of the relative density, BSDFs lying in denser soil show higher MBR values compared to these in loose soil with the varying width.

Table 4.23. MBR values for S_1 considering the changing width.

S_1 (loose sand)					
D (m)	B (m)	Aspect ratio (D/B)	BSDF	Embedded Foundation	MBR
			Bearing Capacity (kPa)	Bearing Capacity (kPa)	
20	4	5.00	4077.91	5670.00	0.72
20	8	2.50	3455.80	5006.35	0.69
20	12	1.67	3341.22	5110.99	0.65
20	16	1.25	3332.41	4933.20	0.68
20	20	1.00	3342.63	4702.16	0.71
20	30	0.67	3243.46	3954.00	0.82
20	40	0.50	2841.88	3110.21	0.91

Table 4.24. MBR values for S_2 considering the changing width.

S_2 (medium dense sand)					
D (m)	B (m)	Aspect ratio (D/B)	BSDF	Embedded Foundation	MBR
			Bearing Capacity (kPa)	Bearing Capacity (kPa)	
20	4	5.00	5456.01	7649.72	0.71
20	8	2.50	4639.82	6723.39	0.69
20	12	1.67	4572.27	6862.38	0.67
20	16	1.25	4508.67	6442.42	0.70
20	20	1.00	4456.74	5933.54	0.75
20	30	0.67	4107.92	4805.00	0.85
20	40	0.50	3435.96	3650.21	0.94

Table 4.25. MBR values for S_3 considering the changing width.

S_3 (dense sand)					
D (m)	B (m)	Aspect ratio (D/B)	BSDF	Embedded Foundation	MBR
			Bearing Capacity (kPa)	Bearing Capacity (kPa)	
20	4	5.00	7039.45	9840.79	0.72
20	8	2.50	5967.22	8667.09	0.69
20	12	1.67	5897.06	8725.02	0.68
20	16	1.25	5737.37	7929.98	0.72
20	20	1.00	5582.58	7082.34	0.79
20	30	0.67	4890.95	5600.17	0.87
20	40	0.50	3962.96	4100.54	0.97

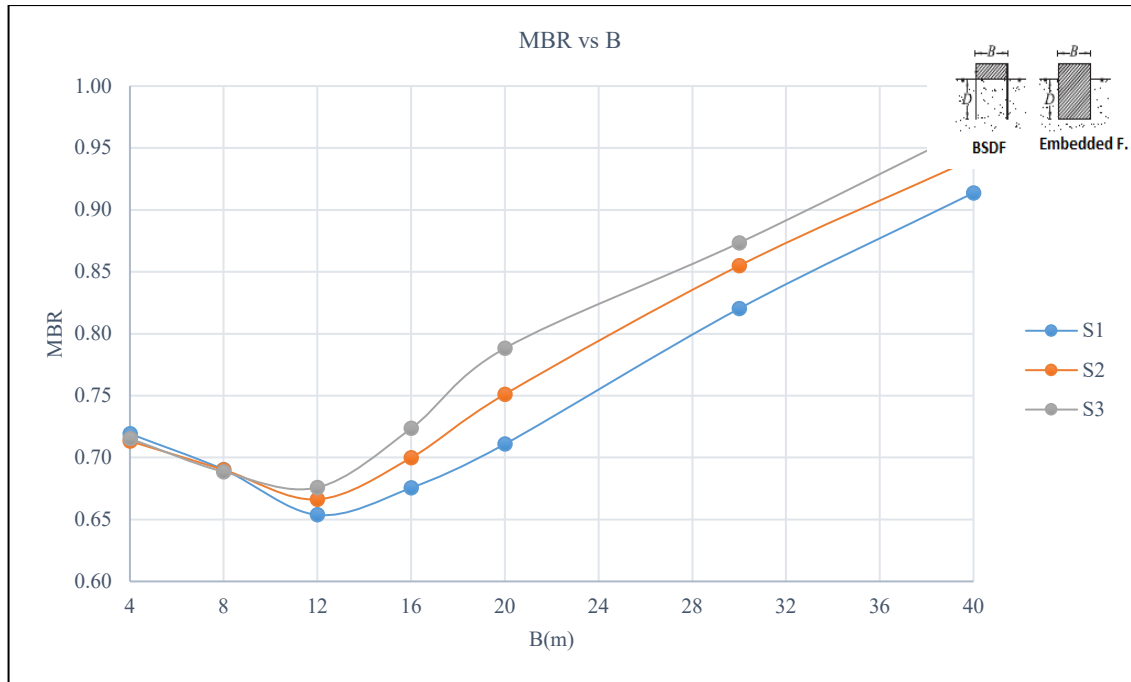


Figure 4.20. MBR vs B plots for S₁, S₂ and S₃.

Table 4.24, 4.25 and 4.26 present the data about MSR values for the changing widths for S₁, S₂ and S₃. Figure 4.19 displays that MSR values increase consistently as width increases. As of the impact of the relative density, BSDFs lying in denser soil show lower MSR values compared to these in loose soil.

Table 4.26. MSR values for S₁ considering the changing width.

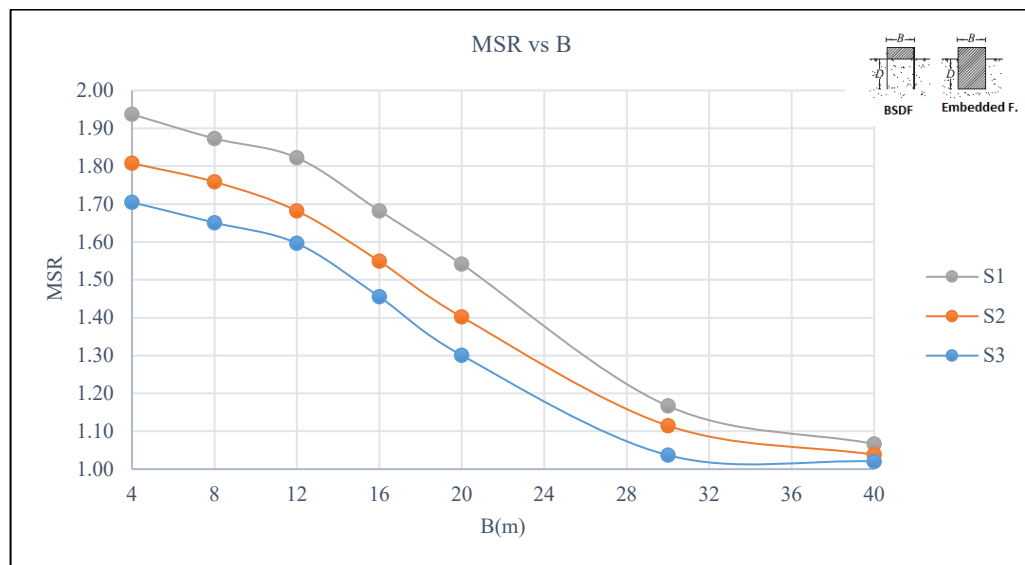
S ₁ (loose sand)					
D (m)	B (m)	Aspect ratio (D/B)	BSDF	Embedded Foundation	MSR
			Settlement (cm)	Settlement (cm)	
20	4	5.00	40.00	20.65	1.94
20	8	2.50	80.00	42.71	1.87
20	12	1.67	120.00	65.84	1.82
20	16	1.25	160.00	95.11	1.68
20	20	1.00	200.00	129.77	1.54
20	30	0.67	300.00	257.13	1.17
20	40	0.50	400.00	375.03	1.07

Table 4.27. MSR values for S_2 considering the changing width.

S_2 (medium dense sand)					
D (m)	B (m)	Aspect ratio (D/B)	BSDF	Embedded Foundation	MSR
			Settlement (cm)	Settlement (cm)	
20	4	5.00	40.00	22.13	1.81
20	8	2.50	80.00	45.49	1.76
20	12	1.67	120.00	71.34	1.68
20	16	1.25	160.00	103.29	1.55
20	20	1.00	200.00	142.67	1.40
20	30	0.67	300.00	269.12	1.11
20	40	0.50	400.00	385.28	1.04

Table 4.28. MSR values for S_3 considering the changing width.

S_3 (dense sand)					
D (m)	B (m)	Aspect ratio (D/B)	BSDF	Embedded Foundation	MSR
			Settlement (cm)	Settlement (cm)	
20	4	5.00	40.00	23.46	1.70
20	8	2.50	80.00	48.46	1.65
20	12	1.67	120.00	75.18	1.60
20	16	1.25	160.00	109.96	1.46
20	20	1.00	200.00	153.78	1.30
20	30	0.67	300.00	289.34	1.04
20	40	0.50	400.00	392.00	1.02

Figure 4.21. MSR vs B plots for S_1 , S_2 and S_3 .

The influence of R_{int} on MBR was explored through multiple simulations using varying R_{int} values: 0.6, 0.8, and one. Table 4.27 shows that MBR values gradually decreases for S_1 and $R_{int}=0.6$. As mentioned above, foundations with the $R_{int}=0.8$ have had a similar trend with the foundations of $R_{int}=0.6$. However, they show a relatively less sharp decrease. On the contrary, as shown in Table 4.28, MBR values for $R_{int}=1$ remain almost the same, fluctuating between 0.90 and 0.86. Figure 4.22 displays that the effect of surface roughness on MBR values decreases as R_{int} values increases.

Table 4.29. MBR vs D plots for $R_{int}=0.6$.

$R_{int}=0.6$					
D (m)	B (m)	Aspect ratio (D/B)	BSDF	Embedded Foundation	MBR
			Bearing Capacity (kPa)	Bearing Capacity (kPa)	
4	8	0.5	1962.17	2293.54	0.86
8	8	1	2075.63	2900.23	0.72
12	8	1.5	2197.52	3397.42	0.65
16	8	2	2390.10	3862.78	0.62
20	8	2.5	2654.93	4439.66	0.60
30	8	3.75	3628.10	6320.73	0.57
40	8	5	4760.08	8015.46	0.59

Table 4.30. MBR vs D plots for $R_{int}=1$.

$R_{int}=1$					
D (m)	B (m)	Aspect ratio (D/B)	BSDF	Embedded Foundation	MBR
			Bearing Capacity (kPa)	Bearing Capacity (kPa)	
4	8	0.5	2107.94	2341.56	0.90
8	8	1	2582.31	3072.13	0.84
12	8	1.5	3173.12	3774.10	0.84
16	8	2	3835.23	4444.66	0.86
20	8	2.5	4570.91	5303.56	0.86
30	8	3.75	6684.73	7788.70	0.86
40	8	5	9062.83	10509.32	0.86

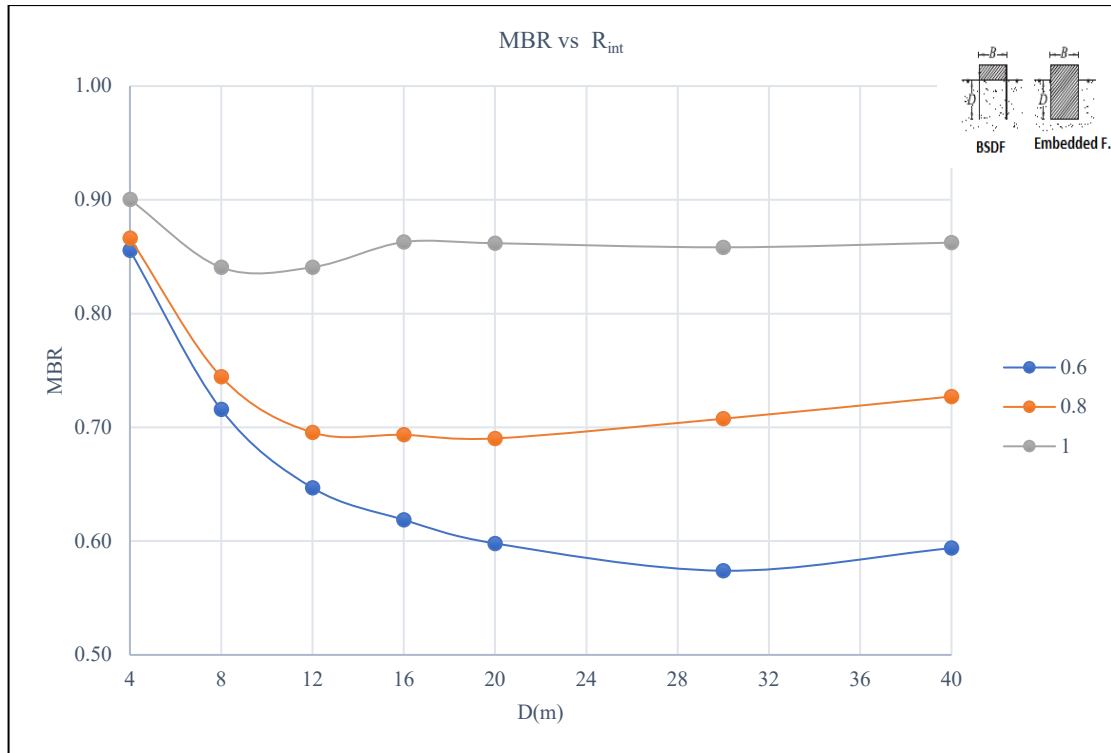


Figure 4.22. MBR vs D plots for $R_{int}=0.6, 0.8$ and 1.0 .

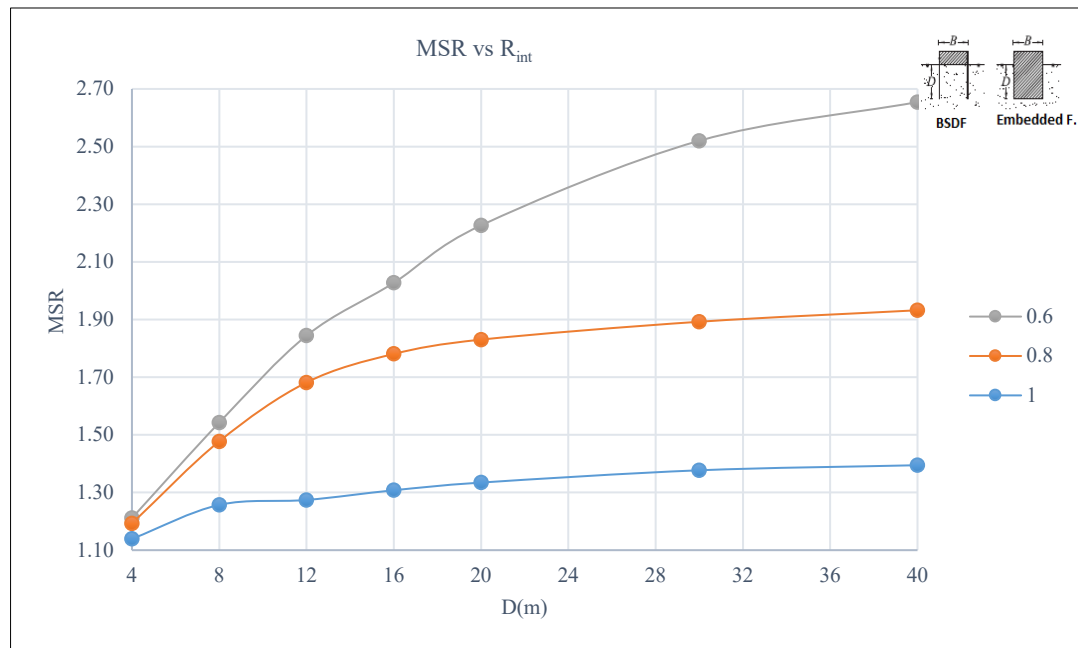
For the foundations with $R_{int}=0.6$, MSR values subsequently rise with the increasing depth. On the other hand, as shown in Table 4.30, MSR values for $R_{int}=1$ increase slightly to 1.27, and then remain nearly unchanged.

Table 4.31. MSR vs D values for $R_{int}=0.6$.

$R_{int}=0.6$					
D (m)	B (m)	Aspect ratio (D/B)	BSDF	Embedded Foundation	MSR
			Settlement (cm)	Settlement (cm)	
4	8	0.5	80.00	66.04	1.21
8	8	1	80.00	51.87	1.54
12	8	1.5	80.00	43.38	1.84
16	8	2	80.00	39.46	2.03
20	8	2.5	80.00	35.93	2.23
30	8	3.75	80.00	31.74	2.52
40	8	5	80.00	30.14	2.65

Table 4.32. MSR vs D values for $R_{int}=1$.

$R_{int}=1$					
D (m)	B (m)	Aspect ratio (D/B)	BSDF	Embedded Foundation	MSR
			Settlement (cm)	Settlement (cm)	
4	8	0.5	80.00	70.23	1.14
8	8	1	80.00	63.64	1.26
12	8	1.5	80.00	62.79	1.27
16	8	2	80.00	61.17	1.31
20	8	2.5	80.00	59.95	1.33
30	8	3.75	80.00	58.09	1.38
40	8	5	80.00	57.36	1.39

Figure 4.23. MSR vs D plots for $R_{int}=0.6, 0.8$ and 1.

Based on the data from this study, MBR values, as the aspect ratio increases, initially move away from one significantly, however, after a certain aspect ratio of 1.5, this trend shifts, and these values start to increase slightly. On the other hand, MSR values increase consistently as the aspect ratio decreases with a lower slope. When it comes to the impact of the relative density, BSDFs lying in denser soil show MBR values closer to unity compared to these in loose soil. Moreover, for aspect ratios below two, MBR values exhibit higher dispersion. When considering the influence of relative density on MSR, BSDFs located in denser soil exhibit MSR values closer to unity compared to those in loose soil. Nevertheless,

it is crucial to acknowledge that geometries sharing identical aspect ratios could manifest different MBR and MSR values.

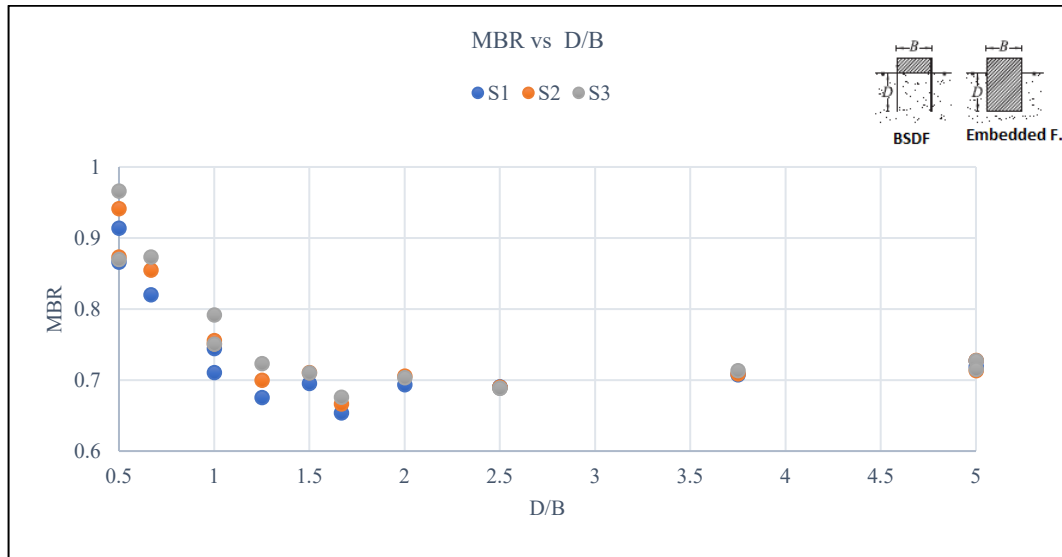


Figure 4.24. MBR vs D/B plots for S₁, S₂ and S₃.

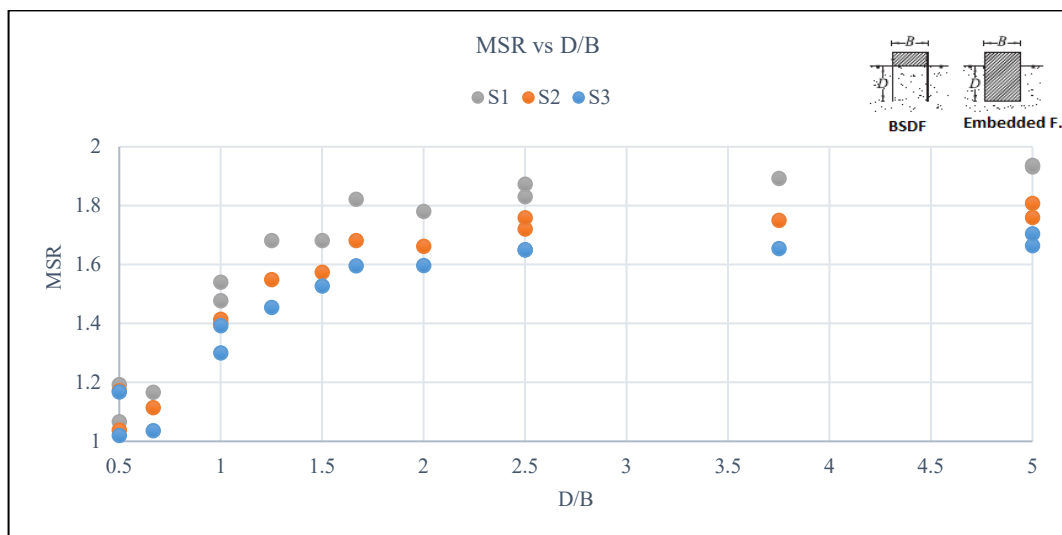


Figure 4.25. MSR vs D/B plots for S₁, S₂ and S₃.

Up to this point, MBR values had been calculated by comparing the bearing capacity determined at a settlement of 10% of the B. However, to represent different load levels, MBR values have also been calculated by comparing the bearing pressures at settlements of 1.25%, 2.5% and 5% of the B.

Table 4.31 and Figure 4.26 depict the effect of different load levels on MBR values for the changing depths. For settlements with magnitudes equal to 1.25% and 2.5% of B, MBR values initially decrease, then increase, and eventually exhibit an oscillatory behavior after a certain point. However, for settlements at 10% and 5% of B, there is a rapid decrease in MBR values, followed by a plateau, and then a slight increase.

Table 4.33. MBR vs D values for different load levels.

D (m)	B (m)	D/B	MBR			
			s=0.0125B	s=0.025B	s=0.05B	s=0.1B
4	8	0.5	0.66	0.73	0.80	0.87
8	8	1	0.56	0.61	0.67	0.74
12	8	1.5	0.61	0.60	0.63	0.70
16	8	2	0.73	0.70	0.67	0.69
20	8	2.5	0.72	0.70	0.68	0.69
30	8	3.75	0.74	0.73	0.71	0.71
40	8	5	0.77	0.75	0.72	0.73

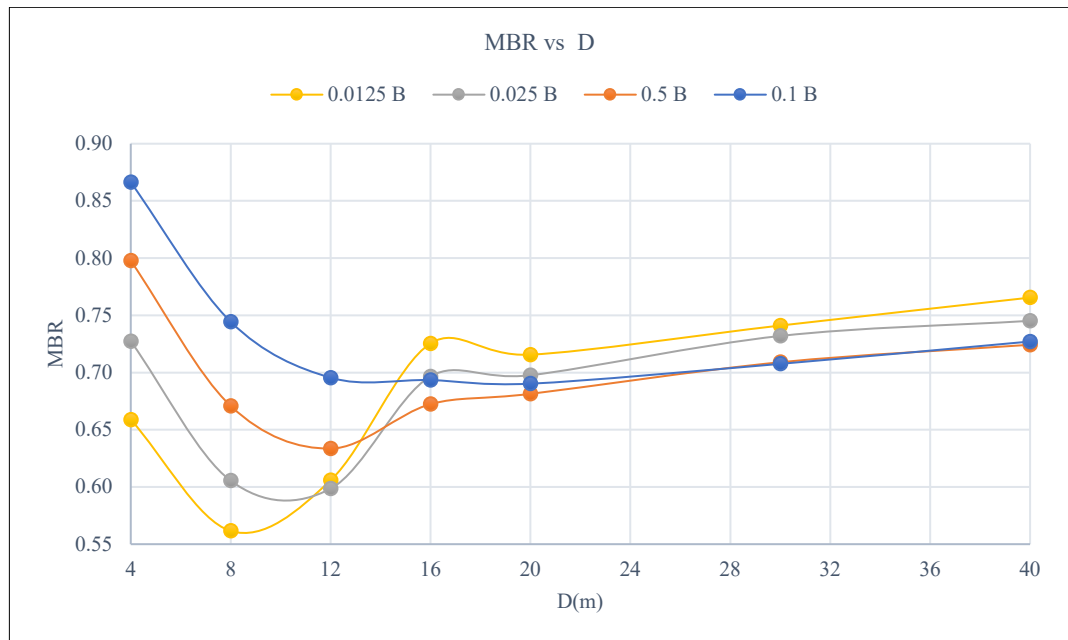


Figure 4.26. MBR vs D plots for different load levels.

Table 4.32 and Figure 4.27 illustrate the effect of different load levels on MBR values for the changing widths. For all load levels, there is a slight increase in MBR values initially as width increases, but after reaching a minimum value, MSR values begin to increase.

Additionally, Figure 4.28 shows the MBR values at different load levels for the changing aspect ratios.

Table 4.34. MBR vs B values for different load levels.

D (m)	B (m)	D/B	MBR			
			s=0.0125B	s=0.025B	s=0.05B	s=0.1B
20	4	5.00	0.71	0.70	0.70	0.72
20	8	2.50	0.72	0.70	0.68	0.69
20	12	1.67	0.67	0.64	0.63	0.65
20	16	1.25	0.66	0.64	0.65	0.68
20	20	1.00	0.67	0.67	0.68	0.71
20	30	0.67	0.75	0.78	0.79	0.82
20	40	0.50	0.86	0.87	0.89	0.91

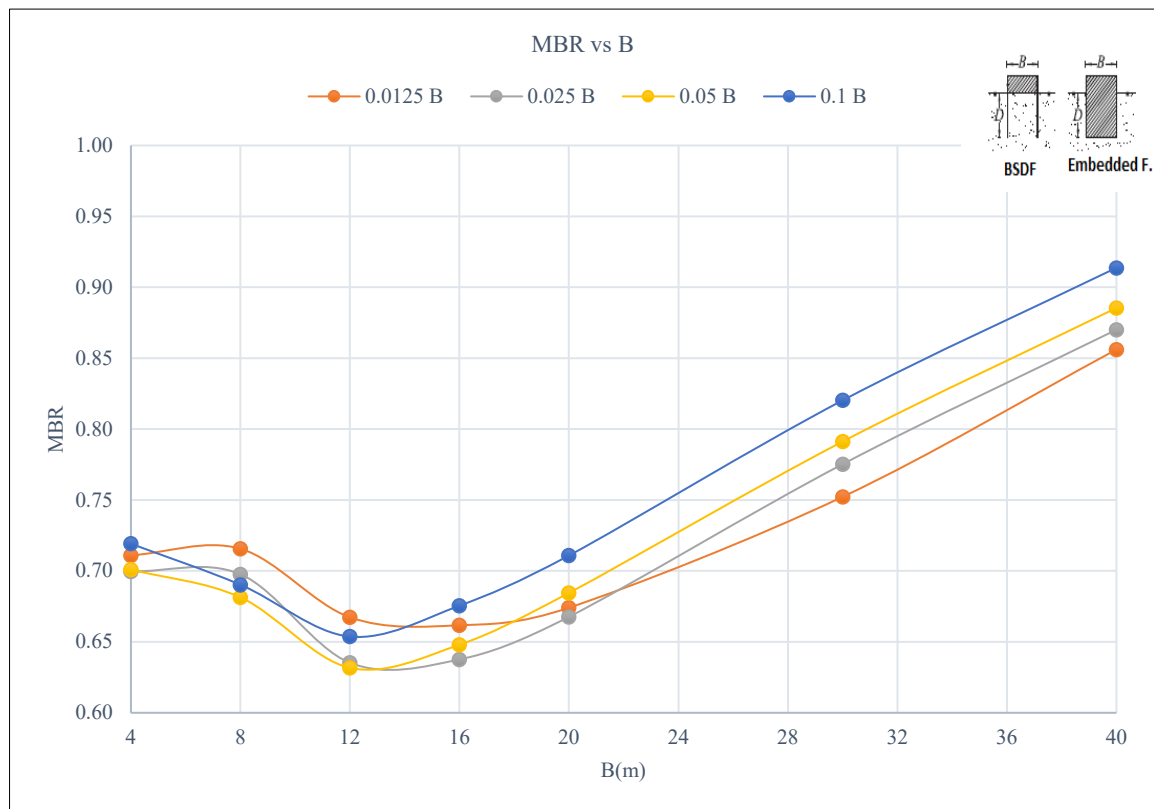


Figure 4.27. MBR vs B plots for different load levels.

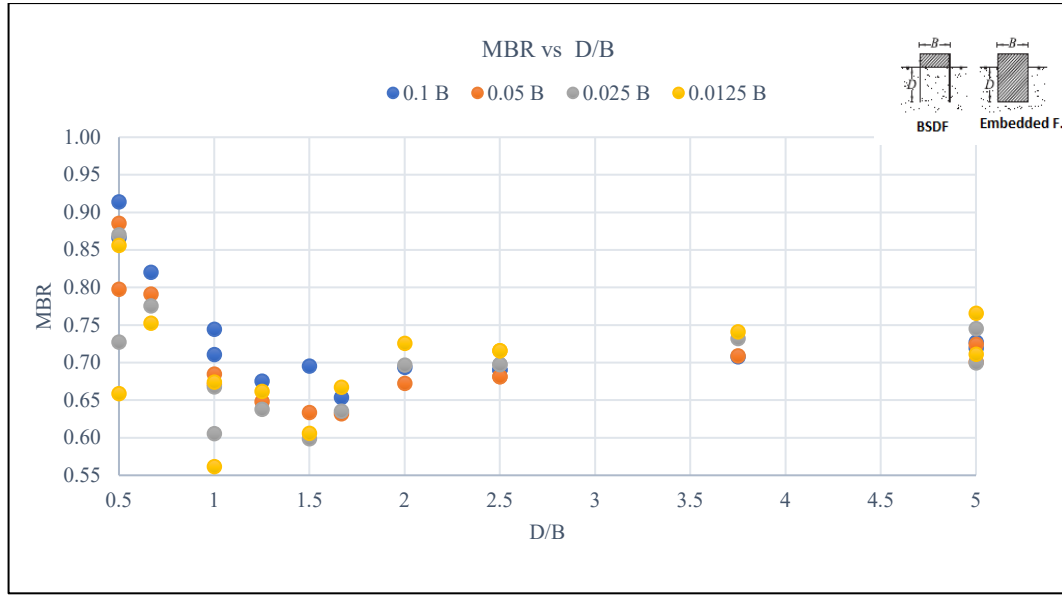


Figure 4.28. MBR vs D/B plots for different load levels.

4.3. Comparison of BSDFs and CPRFs

CPRFs exhibit variations in pile positions, pile numbers, pile lengths, raft-soil stiffness ratios, and load distribution on the raft. Consequently, an infinite number of CPRF configurations can be established for comparison with BSDFs. To facilitate a meaningful comparison between BSDFs and CPRFs, a key parameter, which is the concrete volume of the deep foundations has been considered. Specifically, deep foundations with comparable or identical concrete volumes will be assessed using the efficiency concept.

Efficiency denotes a state in which a given system or apparatus functions optimally. This section is dedicated to the comparative evaluation of the load-settlement behavior of BSDFs and CPRFs, utilizing a quantitative methodology to formulate a comprehensive understanding of efficiency.

P (%) represents the “efficiency” defined as

$$P (\%) = \frac{P_{bsdf}^{50} - P_{cprf}^{50}}{P_{cprf}^{50}}, \quad (5.5)$$

where P_{bsdf}^{50} and P_{cprf}^{50} are the applied loads for a 50 mm settlement on a BSDF and CPRF, respectively.

The load capacity has been computed in accordance with a foundation settlement of 50 mm, aligning with the allowable serviceability limit.

4.3.1. Numerical Modeling

For CPRFs, since 2.5-3 d pile spacing is recommended for the best performance in the literature, the pile spacing has been selected as 3 d. When piles are positioned closely to one another, a reasonable assumption is that the stresses transmitted by the piles to the soil will overlap, leading to a reduction in the load-bearing capacity of the individual piles. The diameter of the piles and box thickness have been chosen as 1 m and 0.6 m, respectively, as shown in Figure 4.29. In order to eliminate different settlements and focus on uniform settlements, the raft thickness was taken as 2 m to attain a rigid foundation. The sand and concrete surfaces were regarded as rough. Thus, no reduction was attributed to the resistance of the interface between sand and concrete.

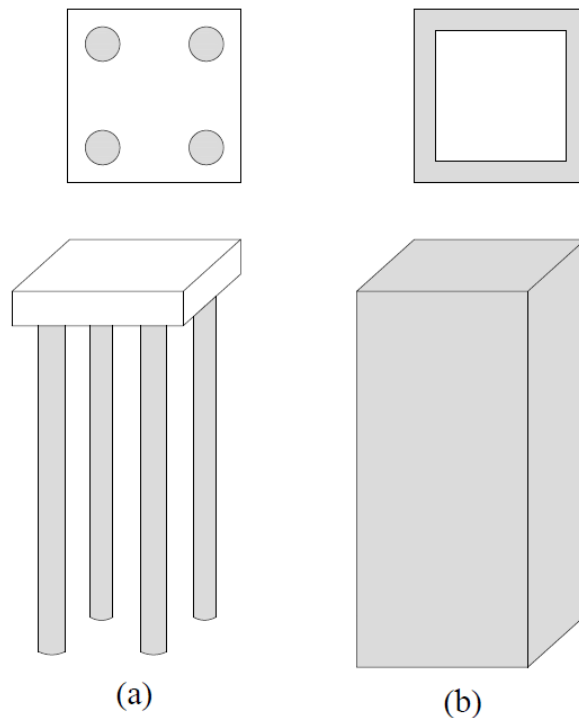


Figure 4.29. CPRF and BSDF configurations.

The detailed assumptions and parameters for numerical analyses of both BSDFs and CPRFs are detailed in Section 4.1.1 and Section 3.2.

4.3.2. Foundations of the Same Volume and Raft Widths

CPRF configurations featuring 4 piles, 25 piles, and 49 piles, as depicted in Figure 4.30, were compared to BSDFs having identical volumes to assess their efficiency. A total of 180 distinct models, encompassing various CPRF aspect ratios ranging from 1 to 4, were simulated, with three distinct soil types for each, allowing for further comparison and interpretation.

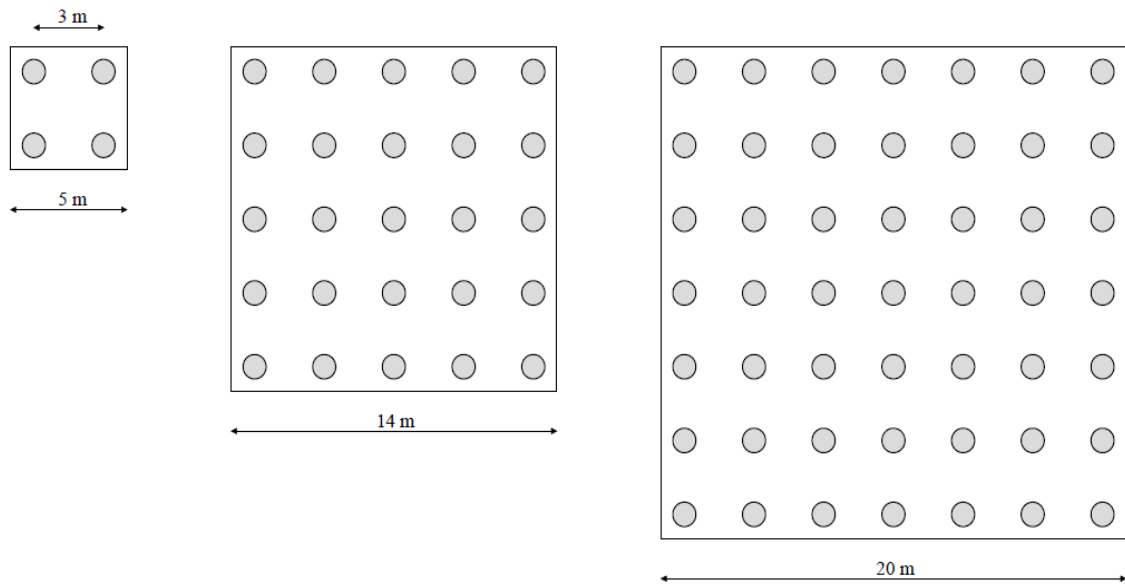


Figure 4.30. Selected CPRF configurations.

It is important to note that in this section, the compared deep foundations have the same raft volume and geometry, while the volume of walls or piles is adjusted to achieve the same total volume.

p_{bsdf}^{50} and p_{cprf}^{50} values were determined based on a uniform settlement of 50 mm and Table 4.33, 4.34 and 4.35 present the P values for the constant raft width of 5 m and changing aspect ratios for S_1 , S_2 and S_3 .

As the aspect ratios of BSDFs increase, efficiency experiences an upward surge, reaching its peak at 6.71% for the BSDF configuration featuring a width of 5 m and a depth of 1.5 m. Conversely, other BSDF configurations exhibit negative efficiency values, which

diminishes their desirability in comparison to the representative CPRF. Figure 4.31 displays that there is a decrease in P values initially as depth of BSDFs increases, but afterwards P values begin to increase. Furthermore, as the aspect ratios of compared CPRFs increase, P values decrease.

As of the impact of the soil density, BSDFs lying in denser soil show higher P values compared to these in loose soil.

Table 4.35. Efficiency values of deep foundations with 5 m raft width in S₁.

Type	D (m)	B (m)	D/B	V (m ³)	V%	P ⁵⁰ (MN)	P (%)
CPRF	5	5	1.00	15.71		6.21	
BSDF	1.5	5	0.30	15.84	0.8%	6.62	6.46%
BSDF	1.93	4	0.48	15.75	0.2%	5.52	-11.10%
BSDF	2.73	3	0.91	15.72	0.1%	5.45	-12.21%
BSDF	4.7	2	2.35	15.79	0.5%	5.74	-7.70%
CPRF	10	5	2.00	31.42		11.33	
BSDF	3	5	0.60	31.68	0.8%	7.31	-35.51%
BSDF	3.85	4	0.96	31.42	0.0%	6.81	-39.92%
BSDF	5.5	3	1.83	31.68	0.8%	7.31	-35.46%
BSDF	9.4	2	4.70	31.58	0.5%	8.81	-22.29%
CPRF	15	5	3.00	47.12		18.54	
BSDF	4.5	5	0.90	47.52	0.8%	9.04	-51.26%
BSDF	5.8	4	1.45	47.33	0.4%	9.01	-51.43%
BSDF	8.25	3	2.75	47.52	0.8%	9.54	-48.53%
BSDF	14.1	2	7.05	47.38	0.5%	13.07	-29.51%
CPRF	20	5	4.00	62.83		27.08	
BSDF	6.00	5	1.20	63.36	0.8%	11.12	-58.95%
BSDF	7.70	4	1.93	62.83	0.0%	11.00	-59.39%
BSDF	11	3	3.67	63.36	0.8%	12.75	-52.93%
BSDF	18.75	2	9.38	63.00	0.3%	18.81	-30.54%

Table 4.36. Efficiency values of deep foundations with 5 m raft width in S₂.

Type	D (m)	B (m)	D/B	V (m ³)	V%	P ⁵⁰ (MN)	P (%)
CPRF	5	5	1.00	15.71		8.56	
BSDF	1.5	5	0.30	15.84	0.8%	9.13	6.67%
BSDF	1.93	4	0.48	15.75	0.2%	7.72	-9.82%
BSDF	2.73	3	0.91	15.72	0.1%	7.58	-11.47%
BSDF	4.7	2	2.35	15.79	0.5%	7.94	-7.24%
CPRF	10	5	2.00	31.42		14.52	
BSDF	3	5	0.60	31.68	0.8%	9.86	-32.12%
BSDF	3.85	4	0.96	31.42	0.0%	9.28	-36.13%
BSDF	5.5	3	1.83	31.68	0.8%	9.90	-31.82%
BSDF	9.4	2	4.70	31.58	0.5%	11.64	-19.84%
CPRF	15	5	3.00	47.12		22.68	
BSDF	4.5	5	0.90	47.52	0.8%	11.93	-47.38%
BSDF	5.8	4	1.45	47.33	0.4%	11.97	-47.21%
BSDF	8.25	3	2.75	47.52	0.8%	12.58	-44.52%
BSDF	14.1	2	7.05	47.38	0.5%	16.49	-27.28%
CPRF	20	5	4.00	62.83		32.18	
BSDF	6.00	5	1.20	63.36	0.8%	14.40	-55.26%
BSDF	7.70	4	1.93	62.83	0.0%	14.27	-55.65%
BSDF	11	3	3.67	63.36	0.8%	16.26	-49.48%
BSDF	18.75	2	9.38	63.00	0.3%	23.06	-28.35%

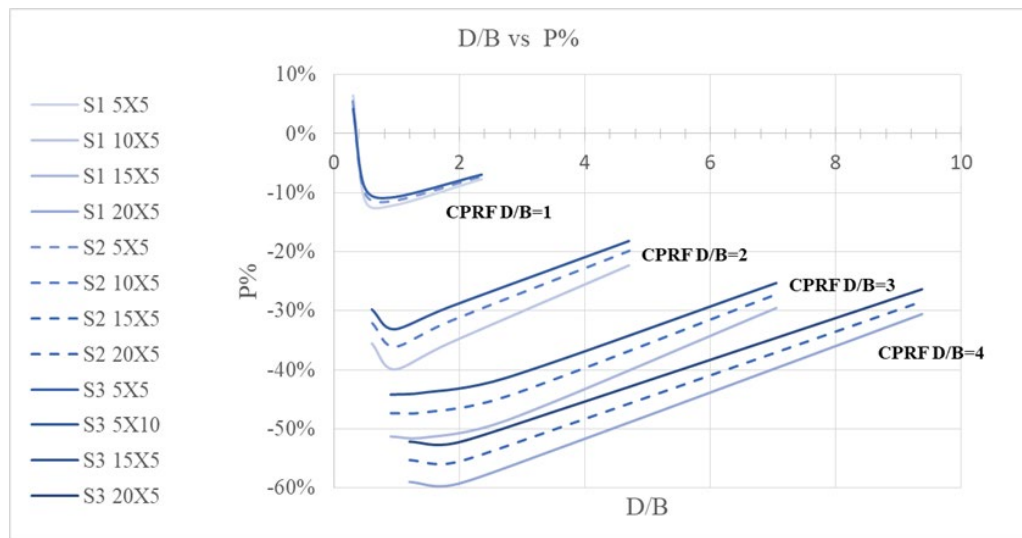
Figure 4.31. Efficiency plots of 5 m width raft in S₁, S₂ and S₃.

Table 4.37. Efficiency values of deep foundations with 5 m raft width in S_3 .

Type	D (m)	B (m)	D/B	V (m ³)	V%	P ⁵⁰ (MN)	P (%)
CPRF	5	5	1.00	15.71		11.17	
BSDF	1.5	5	0.30	15.84	0.8%	11.92	6.71%
BSDF	1.93	4	0.48	15.75	0.2%	10.16	-9.08%
BSDF	2.73	3	0.91	15.72	0.1%	9.96	-10.81%
BSDF	4.7	2	2.35	15.79	0.5%	10.40	-6.93%
CPRF	10	5	2.00	31.42		18.05	
BSDF	3	5	0.60	31.68	0.8%	12.68	-29.76%
BSDF	3.85	4	0.96	31.42	0.0%	12.07	-33.10%
BSDF	5.5	3	1.83	31.68	0.8%	12.75	-29.35%
BSDF	9.4	2	4.70	31.58	0.5%	14.77	-18.17%
CPRF	15	5	3.00	47.12		27.08	
BSDF	4.5	5	0.90	47.52	0.8%	15.12	-44.18%
BSDF	5.8	4	1.45	47.33	0.4%	15.22	-43.80%
BSDF	8.25	3	2.75	47.52	0.8%	15.89	-41.31%
BSDF	14.1	2	7.05	47.38	0.5%	20.23	-25.28%
CPRF	20	5	4.00	62.83		37.46	
BSDF	6.00	5	1.20	63.36	0.8%	17.93	-52.13%
BSDF	7.70	4	1.93	62.83	0.0%	17.84	-52.38%
BSDF	11	3	3.67	63.36	0.8%	20.05	-46.49%
BSDF	18.75	2	9.38	63.00	0.3%	27.60	-26.34%

Table 4.36, 4.37 and 4.38 present the data about P values for the constant raft width of 14 m and changing aspect ratios for S_1 , S_2 and S_3 . As the aspect ratios of BSDFs increase, efficiency experiences an upward surge, reaching its peak at 23.56% for the BSDF configuration featuring a width of 8 m and a depth of 62 m. Furthermore, P values tend to exhibit positive values, as the aspect ratios of compared CPRFs increase.

BSDF configurations of the compared CPRF with the aspect ratio of one see negative efficiency values for all BSDF aspect ratios and soil types, which diminishes their efficiency in comparison to the representative CPRF.

Figure 4.32 displays that P values decrease as depth of BSDFs increases and BSDFs lying in denser soil show higher P values compared to these in loose soil.

Table 4.38. Efficiency values of deep foundations with 14 m raft width in S_1 .

Type	D (m)	B (m)	D/B	V (m ³)	V%	P ⁵⁰ (MN)	P (%)
CPRF	14	14	1.00	274.89		57.47	
BSDF	8.55	14	0.61	274.97	0.0%	50.07	-12.88%
BSDF	10.05	12	0.84	274.97	0.0%	50.18	-12.69%
BSDF	12.2	10	1.22	275.23	0.1%	54.39	-5.36%
BSDF	15.5	8	1.94	275.28	0.1%	56.06	-2.44%
CPRF	28	14	2.00	549.78		118.90	
BSDF	17.1	14	1.22	549.94	0.0%	89.66	-24.59%
BSDF	20.1	12	1.68	549.94	0.0%	93.27	-21.55%
BSDF	24.4	10	2.44	550.46	0.1%	108.31	-8.90%
BSDF	31	8	3.88	550.56	0.1%	122.71	3.21%
CPRF	42	14	3.00	824.67		180.94	
BSDF	25.65	14	1.83	824.90	0.0%	128.81	-28.81%
BSDF	30.25	12	2.52	827.64	0.4%	143.53	-20.67%
BSDF	36.55	10	3.66	824.57	0.0%	167.13	-7.63%
BSDF	46.5	8	5.81	825.84	0.1%	203.99	12.74%
CPRF	56	14	4.00	1099.56		224.21	
BSDF	34.25	14	2.45	1101.48	0.2%	180.34	-29.00%
BSDF	40.25	12	3.35	1101.24	0.2%	191.51	-14.58%
BSDF	48.75	10	4.88	1099.80	0.0%	226.34	0.95%
BSDF	62	8	7.75	1101.12	0.1%	270.44	20.62%

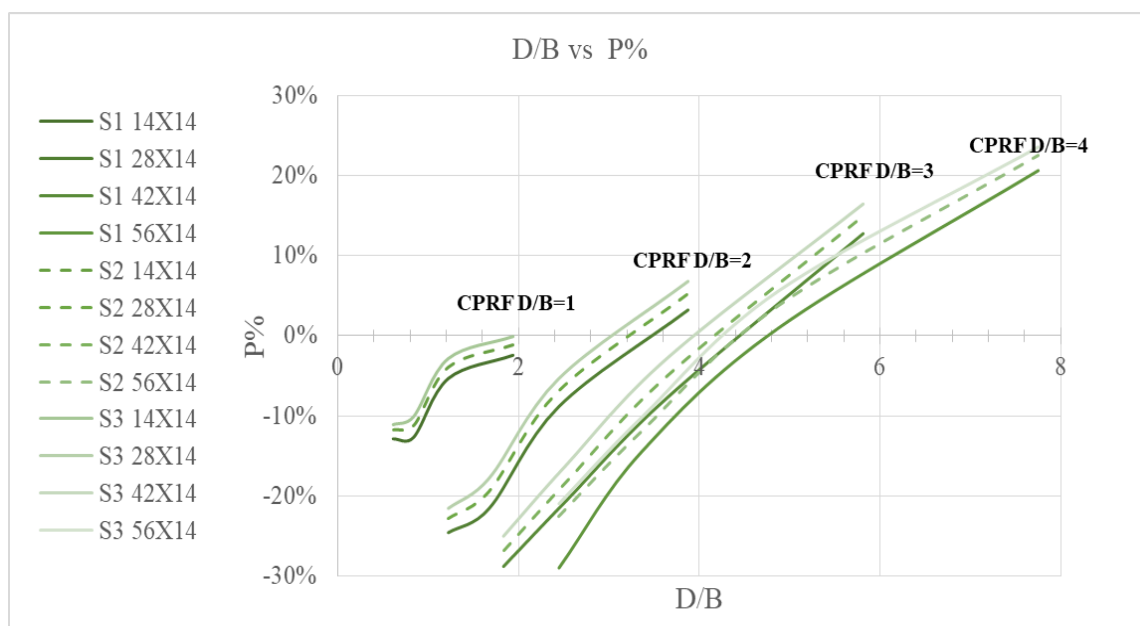


Figure 4.32. Efficiency plots of 14 m width raft in S₁, S₂ and S₃.

Table 4.39. Efficiency values of deep foundations with 14 m raft width in S₂.

Type	D (m)	B (m)	D/B	V (m ³)	V%	P ⁵⁰ (MN)	P (%)
CPRF	14	14	1.00	274.89		67.58	
BSDF	8.55	14	0.61	274.97	0.0%	59.60	-11.80%
BSDF	10.05	12	0.84	274.97	0.0%	60.00	-11.22%
BSDF	12.2	10	1.22	275.23	0.1%	64.91	-3.95%
BSDF	15.5	8	1.94	275.28	0.1%	66.78	-1.18%
CPRF	28	14	2.00	549.78		131.07	
BSDF	17.1	14	1.22	549.94	0.0%	101.08	-22.88%
BSDF	20.1	12	1.68	549.94	0.0%	105.58	-19.45%
BSDF	24.4	10	2.44	550.46	0.1%	121.98	-6.94%
BSDF	31	8	3.88	550.56	0.1%	137.96	5.26%
CPRF	42	14	3.00	824.67		191.14	
BSDF	25.65	14	1.83	824.90	0.0%	139.99	-26.76%
BSDF	30.25	12	2.52	827.64	0.4%	156.30	-18.22%
BSDF	36.55	10	3.66	824.57	0.0%	181.88	-4.84%
BSDF	46.5	8	5.81	825.84	0.1%	219.72	14.95%
CPRF	56	14	4.00	1099.56		229.62	
BSDF	34.25	14	2.45	1101.48	0.2%	177.65	-22.63%
BSDF	40.25	12	3.35	1101.24	0.2%	201.85	-12.09%
BSDF	48.75	10	4.88	1099.80	0.0%	238.14	3.71%
BSDF	62	8	7.75	1101.12	0.1%	281.31	22.51%

Table 4.40. Efficiency values of deep foundations with 14 m raft width in S₃.

Type	D (m)	B (m)	D/B	V (m ³)	V%	P ⁵⁰ (MN)	P (%)
CPRF	14	14	1.00	274.89		77.23	
BSDF	8.55	14	0.61	274.97	0.0%	68.65	-11.11%
BSDF	10.05	12	0.84	274.97	0.0%	69.43	-10.10%
BSDF	12.2	10	1.22	275.23	0.1%	75.02	-2.86%
BSDF	15.5	8	1.94	275.28	0.1%	77.15	-0.10%
CPRF	28	14	2.00	549.78		142.13	
BSDF	17.1	14	1.22	549.94	0.0%	111.49	-21.56%
BSDF	20.1	12	1.68	549.94	0.0%	116.86	-17.78%
BSDF	24.4	10	2.44	550.46	0.1%	134.45	-5.40%
BSDF	31	8	3.88	550.56	0.1%	151.78	6.79%
CPRF	42	14	3.00	824.67		200.18	
BSDF	25.65	14	1.83	824.90	0.0%	150.07	-25.03%
BSDF	30.25	12	2.52	827.64	0.4%	167.71	-16.22%
BSDF	36.55	10	3.66	824.57	0.0%	194.77	-2.70%
BSDF	46.5	8	5.81	825.84	0.1%	233.09	16.44%
CPRF	56	14	4.00	1099.56		235.21	
BSDF	34.25	14	2.45	1101.48	0.2%	185.85	-20.98%
BSDF	40.25	12	3.35	1101.24	0.2%	211.03	-10.28%
BSDF	48.75	10	4.88	1099.80	0.0%	248.33	5.58%
BSDF	62	8	7.75	1101.12	0.1%	290.62	23.56%

Table 4.39, 4.40 and 4.41 present the data about *P* values for the constant raft width of 20 m and changing aspect ratios for S₁, S₂ and S₃. As the aspect ratios of BSDFs increase, efficiency experiences an upward surge, reaching its peak at 24.26% for the BSDF configuration featuring a width of 20 m and a depth of 96 m. Furthermore, *P* values tend to exhibit negative values, as the aspect ratios of compared CPRFs increase.

BSDF configurations of the compared CPRF with the aspect ratio of 1 enjoy positive efficiency values for BSDF aspect ratios higher than 1.2 and all soil types, which increase their preference in comparison to the representative CPRF. “Figure 4.33” displays that *P* values increase as depth of BSDFs increases and BSDFs lying in looser soil show higher *P* values compared to these in dense soil.

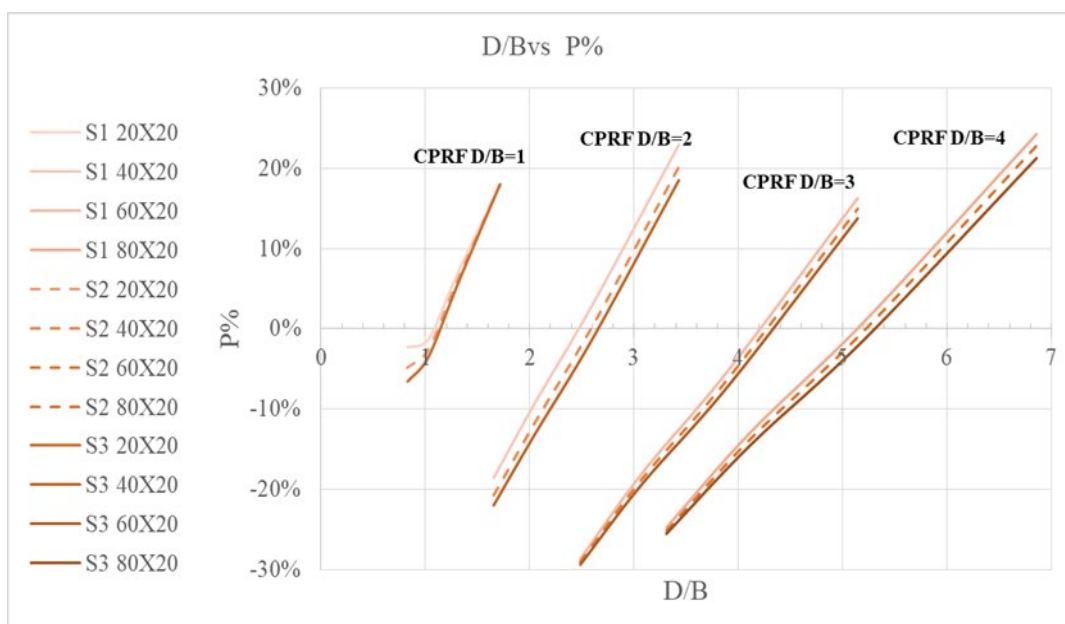


Figure 4.33. Efficiency plots of 20 m width raft in S₁, S₂ and S₃.

Table 4.41. Efficiency values of deep foundations with 20 m raft width in S₁.

Type	D (m)	B (m)	D/B	V (m ³)	V%	P ⁵⁰ (MN)	P (%)
CPRF	20	20	1.00	769.69		111.43	
BSDF	16.6	20	0.83	772.90	181.2%	108.89	-2.27%
BSDF	18.5	18	1.03	772.56	181.0%	109.95	-1.32%
BSDF	20.9	16	1.31	772.46	181.0%	119.05	6.84%
BSDF	24	14	1.71	771.84	180.8%	131.44	17.97%
CPRF	40	20	2.00	1539.38		197.57	
BSDF	33.1	20	1.66	1541.14	180.3%	160.95	-18.54%
BSDF	37	18	2.06	1545.12	181.0%	179.61	-9.09%
BSDF	41.8	16	2.61	1544.93	181.0%	204.04	3.27%
BSDF	48	14	3.43	1543.68	180.8%	242.88	22.93%
CPRF	60	20	3.00	2309.07		257.40	
BSDF	49.7	20	2.49	2314.03	180.6%	183.72	-28.62%
BSDF	55.4	18	3.08	2313.50	180.5%	211.19	-17.95%
BSDF	62.5	16	3.91	2310.00	180.1%	244.28	-5.10%
BSDF	72	14	5.14	2315.52	180.8%	299.32	16.29%
CPRF	80	20	4.00	3078.76		257.37	
BSDF	66.2	20	3.31	3082.27	180.3%	193.73	-24.73%
BSDF	74.0	18	4.11	3090.24	181.0%	224.07	-12.94%
BSDF	83.5	16	5.22	3086.16	180.7%	260.19	1.09%
BSDF	96	14	6.86	3087.36	180.8%	319.81	24.26%

Table 4.42. Efficiency values of deep foundations with 20 m raft width in S₂.

Type	D (m)	B (m)	D/B	V (m ³)	V%	P ⁵⁰ (MN)	P (%)
CPRF	20	20	1.00	769.69		122.58	
BSDF	16.6	20	0.83	772.90	181.2%	116.59	-4.89%
BSDF	18.5	18	1.03	772.56	181.0%	119.29	-2.68%
BSDF	20.9	16	1.31	772.46	181.0%	129.95	6.02%
BSDF	24	14	1.71	771.84	180.8%	144.64	18.00%
CPRF	40	20	2.00	1539.38		206.43	
BSDF	33.1	20	1.66	1541.14	180.3%	163.77	-20.67%
BSDF	37	18	2.06	1545.12	181.0%	182.73	-11.48%
BSDF	41.8	16	2.61	1544.93	181.0%	207.87	0.70%
BSDF	48	14	3.43	1543.68	180.8%	248.29	20.27%
CPRF	60	20	3.00	2309.07		261.09	
BSDF	49.7	20	2.49	2314.03	180.6%	185.22	-29.06%
BSDF	55.4	18	3.08	2313.50	180.5%	212.25	-18.70%
BSDF	62.5	16	3.91	2310.00	180.1%	245.04	-6.15%
BSDF	72	14	5.14	2315.52	180.8%	300.05	14.93%
CPRF	80	20	4.00	3078.76		261.00	
BSDF	66.2	20	3.31	3082.27	180.3%	195.41	-25.13%
BSDF	74.0	18	4.11	3090.24	181.0%	225.14	-13.74%
BSDF	83.5	16	5.22	3086.16	180.7%	260.93	-0.03%
BSDF	96	14	6.86	3087.36	180.8%	320.40	22.76%

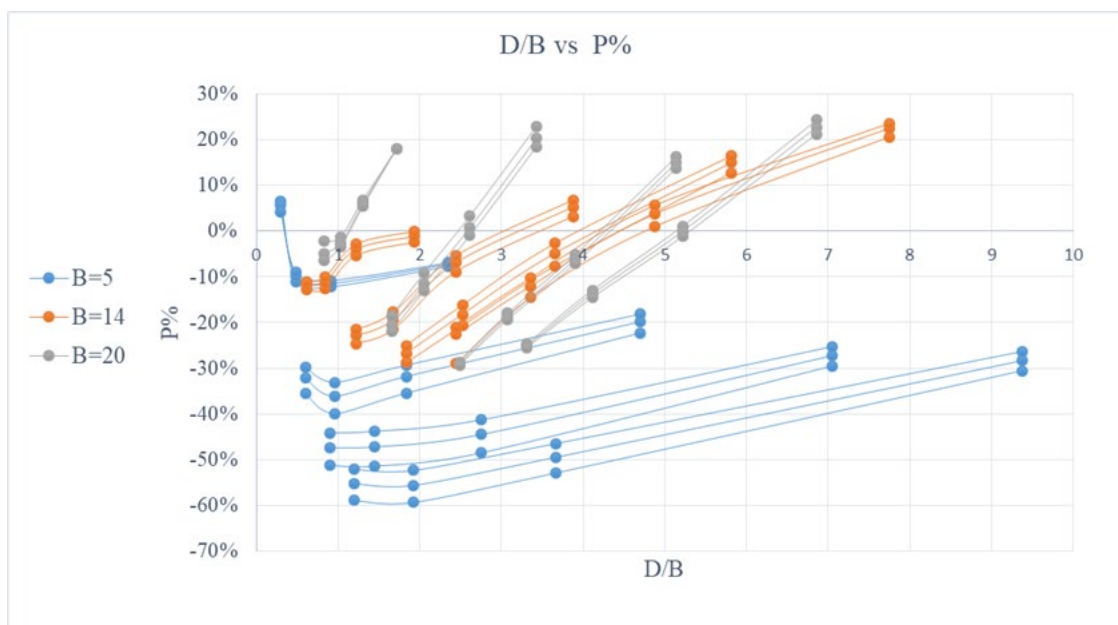


Figure 4.33. Efficiency plots of all simulated BSDFs.

Table 4.43. Efficiency values of deep foundations with 20 m raft width in S₃.

Type	D (m)	B (m)	D/B	V (m ³)	V%	P ⁵⁰ (MN)	P (%)
CPRF	20	20	1.00	769.69		132.52	
BSDF	16.6	20	0.83	772.90	181.2%	123.86	-6.54%
BSDF	18.5	18	1.03	772.56	181.0%	127.84	-3.53%
BSDF	20.9	16	1.31	772.46	181.0%	139.76	5.47%
BSDF	24	14	1.71	771.84	180.8%	156.37	18.00%
CPRF	40	20	2.00	1539.38		214.27	
BSDF	33.1	20	1.66	1541.14	180.3%	167.25	-21.95%
BSDF	37	18	2.06	1545.12	181.0%	186.45	-12.99%
BSDF	41.8	16	2.61	1544.93	181.0%	212.17	-0.98%
BSDF	48	14	3.43	1543.68	180.8%	253.94	18.51%
CPRF	60	20	3.00	2309.07		264.89	
BSDF	49.7	20	2.49	2314.03	180.6%	187.14	-29.35%
BSDF	55.4	18	3.08	2313.50	180.5%	213.76	-19.30%
BSDF	62.5	16	3.91	2310.00	180.1%	246.26	-7.03%
BSDF	72	14	5.14	2315.52	180.8%	301.37	13.77%
CPRF	80	20	4.00	3078.76		264.86	
BSDF	66.2	20	3.31	3082.27	180.3%	197.29	-25.51%
BSDF	74.0	18	4.11	3090.24	181.0%	226.40	-14.52%
BSDF	83.5	16	5.22	3086.16	180.7%	261.82	-1.15%
BSDF	96	14	6.86	3087.36	180.8%	321.18	21.26%

Figure 4.34 illustrates that efficiency of most of the BSDF configurations exhibit negative values. This implies that the simulated CPRF models perform better than the BSDF models with the same volume, making CPRFs more preferable than BSDFs in most cases.

4.3.3. Foundations of the Same Volume and Different Raft Widths

The efficiency of BSDF configurations was assessed by comparing them to CPRFs of equal volumes, featuring 9 piles as shown in Figure 4.25. A total of 15 distinct models were simulated, encompassing various BSDF aspect ratios ranging from 0.81 to 4.68 and a single aspect ratio for the counterpart CPRF which is one. For each aspect ratio, three different soil types were considered, allowing for further analysis and comparison. It is important to note that in this section, the compared deep foundations have different raft volumes and geometries, and the volume of walls or piles is adjusted to achieve equal total volumes.

Table 4.42, 4.43 and 4.44 present the data about P values for BSDFs of the constant total volume and changing aspect ratios for S_1 , S_2 and S_3 . As the aspect ratios of BSDFs increase, efficiency values enjoy a huge increase, starting from -21.94% at the aspect ratio of 0.81 and reaching its peak at 73% at the aspect ratio of 4.68. As of the impact of relative density of the soil, BSDFs lying in looser soil show higher P values compared to these in dense soil shown in Figure 4.35. Overall, this behavior is more pronounced after an aspect ratio of one.

Table 4.44. Efficiency values of deep foundations with 8 m raft width in S_1 .

S_1 (loose sand)									
Type	D (m)	B (m)	D/B	V_r (m ³)	V_w (m ³)	V (m ³)	V%	P^{50} (MN)	$P\%$
CPRF	8	8	1.00	56.55	128.00	184.55		17.46	
BSDF	5.65	7	0.81	86.78	98.00	184.78	0.1%	13.63	-21.94%
BSDF	8.7	6	1.45	112.75	72.00	184.75	0.1%	16.19	-7.27%
BSDF	12.75	5	2.55	134.64	50.00	184.64	0.0%	20.72	18.65%
BSDF	18.7	4	4.68	152.59	32.00	184.59	0.0%	30.21	73.00%

Table 4.45. Efficiency values of deep foundations with 8 m raft width in S_2 .

S_2 (medium dense sand)									
Type	D (m)	B (m)	D/B	V_r (m ³)	V_w (m ³)	V (m ³)	V%	P^{50} (MN)	$P\%$
CPRF	8	8	1.00	56.55	128.00	184.55		22.39	
BSDF	5.65	7	0.81	86.78	98.00	184.78	0.1%	17.83	-20.36%
BSDF	8.7	6	1.45	112.75	72.00	184.75	0.1%	20.55	-8.22%
BSDF	12.75	5	2.55	134.64	50.00	184.64	0.0%	25.48	13.81%
BSDF	18.7	4	4.68	152.59	32.00	184.59	0.0%	35.87	60.19%

Table 4.46. Efficiency values of deep foundations with 8 m raft width in S_3 .

S_3 (dense sand)									
Type	D (m)	B (m)	D/B	V_f (m ³)	V_r (m ³)	V (m ³)	V%	P^{50} (MN)	$P\%$
CPRF	8	8	1.00	56.55	128.00	184.55		27.74	
BSDF	5.65	7	0.81	86.78	98.00	184.78	0.1%	22.32	-19.54%
BSDF	8.7	6	1.45	112.75	72.00	184.75	0.1%	25.21	-9.11%
BSDF	12.75	5	2.55	134.64	50.00	184.64	0.0%	30.56	10.20%
BSDF	18.7	4	4.68	152.59	32.00	184.59	0.0%	41.65	50.16%

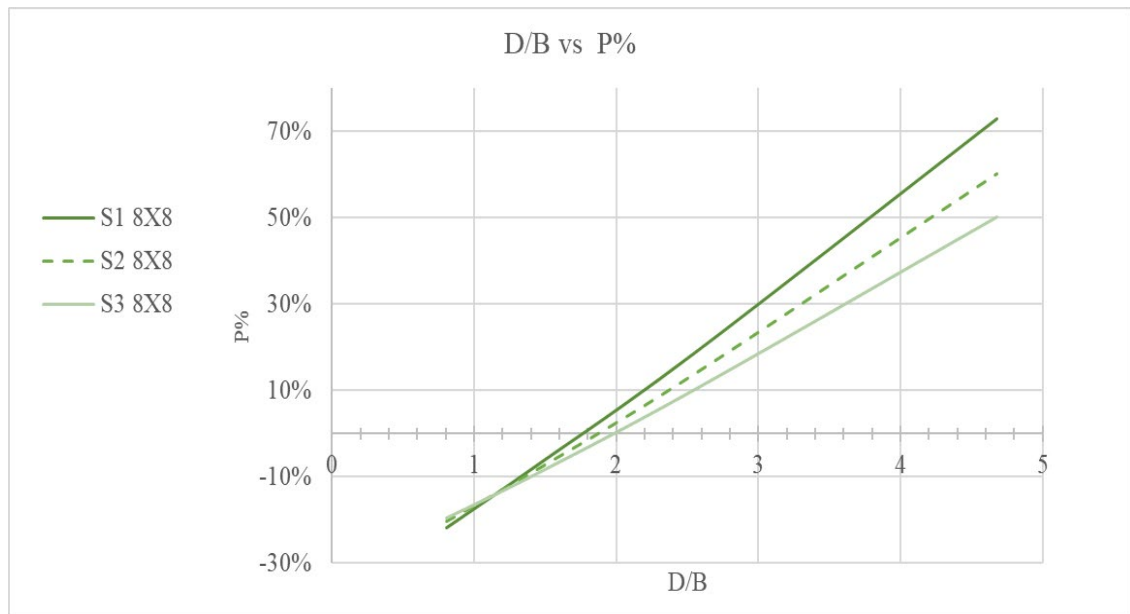


Figure 4.34. Efficiency plots of 8 m width raft in S_1 , S_2 and S_3 .

5. EXPERIMENTAL STUDY

5.1. Experimental Study

The experimental study was conducted to observe the load-settlement behavior and monolithic characteristics of model BSDFs. Additionally, the obtained results were compared with the outcomes of numerical analysis to validate their accuracy.

The test tank was divided into ten layers by drawing lines on the plexiglass with 5 cm intervals. In each layer, the required soil mass, determined according to the intended relative density, was added. Subsequently, the sand was compressed to attain the desired volume. The experimental study steps are shown in Figure 5.1.

The experimental procedure was conducted as outlined below:

- Throughout the experiment, a desired relative density of 35% was accomplished by incorporating 10% of the required soil mass into each layer. Once the desired depth was reached, the box was situated on the soil surface.
- The box was filled with sand to achieve the target relative density of 35%, and then the raft plate was securely fixed in position utilizing screws.
- Following the placement of the BSDF, the soil was further filled until it reached the top level.
- The LVDT transducer was positioned at the edge of the foundation.
- The BSDF underwent a consistent loading rate of 10 mm/min until a displacement of 20 mm was attained. Throughout the process, the LabView software recorded the load-displacement values for further evaluation.

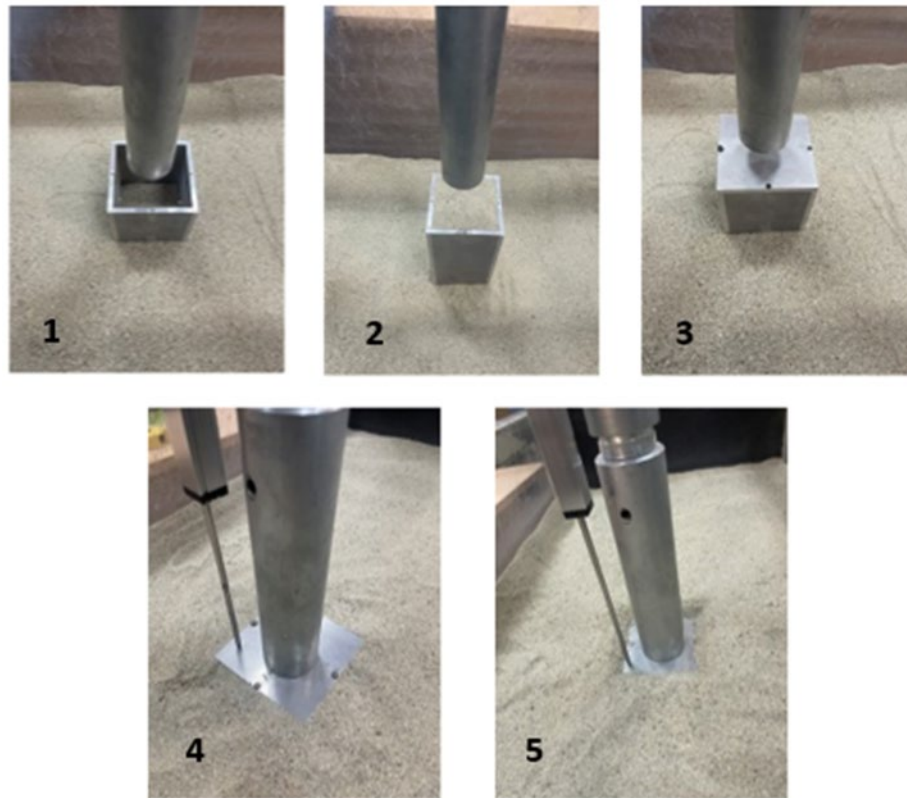


Figure 5.1. The experimental study steps.

After conducting load-settlement tests on model foundations as detailed in Table 3.2, a numerical analysis was performed using PLAXIS 3D to verify the test results and gain insights into the performance of various parametric studies. Numerical simulation model is shown in Figure 5.2. For this analysis, the parameters of the HSS model were calculated based on the formulas provided in 3.2.2, specifically considering a relative density of 35%.

For the analysis, the interfaces between the sand and aluminum plates were assumed to have smooth contact. To account for soil-structure interaction effects in the model, a strength reduction factor (R_{int}) of 0.5 was utilized to represent the interaction between model foundations and the surrounding soil. It is possible to make inferences about the behavior of large-scale BSDFs based on the behavior of small-scale BSDFs through the investigation of the scale effect. Since the primary aim of the experimental study was to validate the numerical simulations, in this study, scale effects were not taken into consideration, and both experimental and numerical models were analyzed as model foundations.

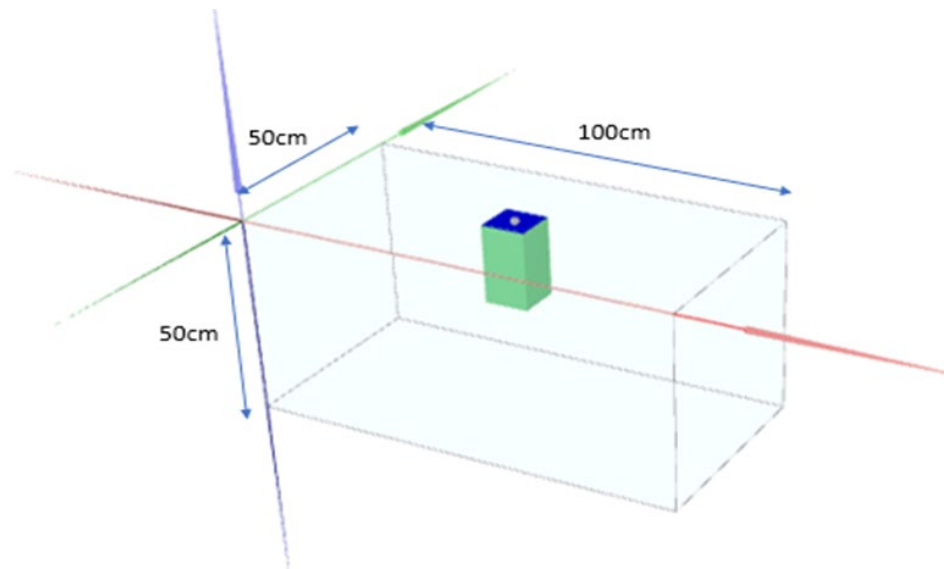


Figure 5.2. Numerical simulation of the experimental study.

5.2. Load-settlement Behavior of BSDFs

Figure 5.1, 5.2, 5.3 and 5.4 present the load-settlement plots obtained through numerical simulations and experimental studies for both BSDFs and their embedded counterparts. The bearing capacity for all cases was calculated as 10% of the foundation width, equivalent to 10 mm.

Table 5.1 illustrates the experimentally and numerically obtained load capacity values for BSDFs and their embedded counterparts. The load capacity difference between BSDFs with 20 cm depth and 10 cm width is 16%. For the embedded counterparts of BSDF 20x10, this difference reaches to 10%. As for BSDFs with 10 cm depth and 10 cm width, the load capacity difference is 18%. The embedded counterparts of BSDF 10x10 have a load capacity difference of 11%.

Table 5.1. Experimentally and numerically obtained load capacity values.

Model Foundation	D (cm)	B (cm)	Load Capacity (kg)		Difference
			Experimental	Numerical	
BSDF 20x10	20	10	164.18	196.07	16%
Embedded 20x10	20	10	222.34	246.16	10%
BSDF 10x10	10	10	144.14	176.51	18%
Embedded 10x10	10	10	170.4	191.54	11%

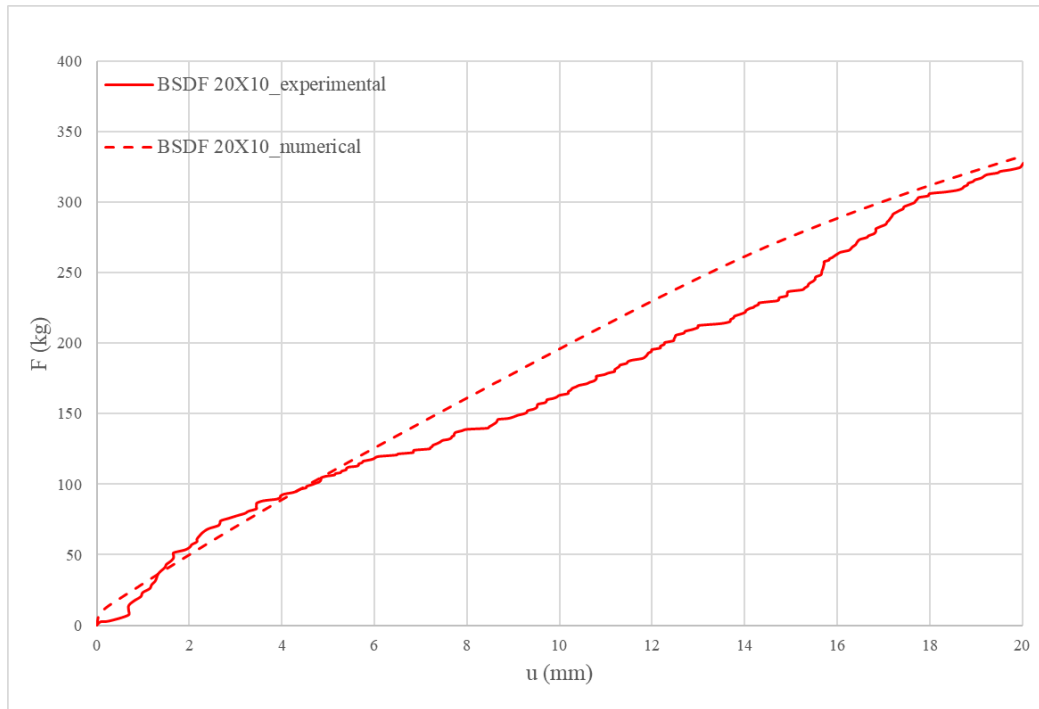


Figure 5.3. Load-settlement behavior of BSDF 20x10.

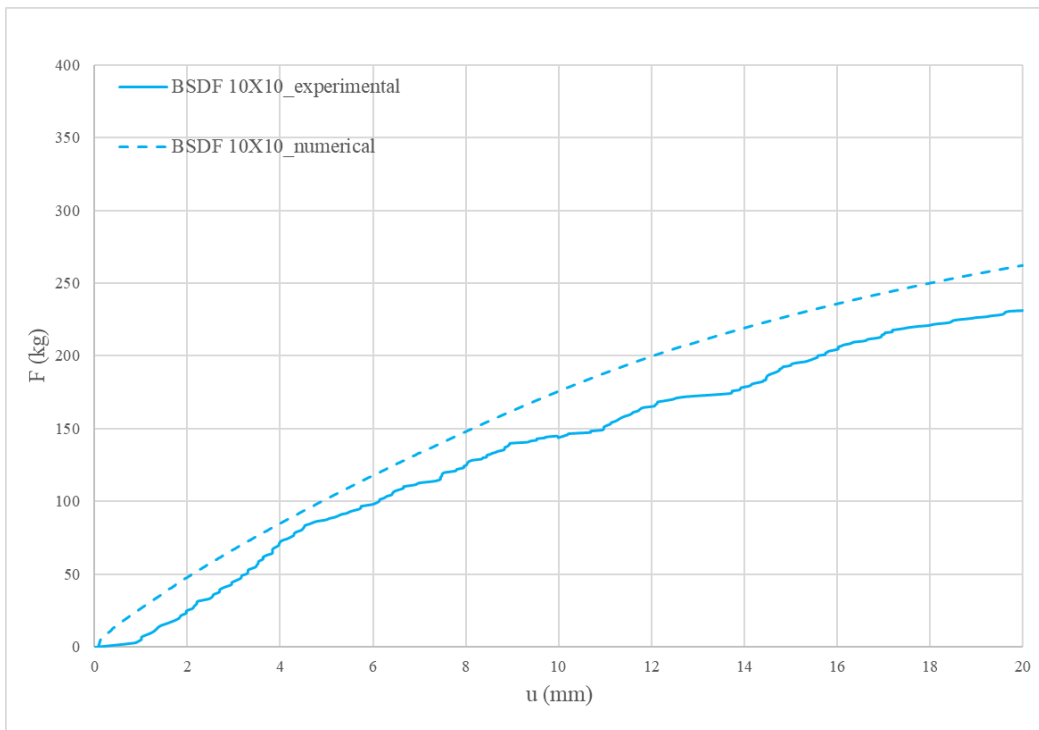


Figure 5.4. Load-settlement behavior of BSDF 10x10.

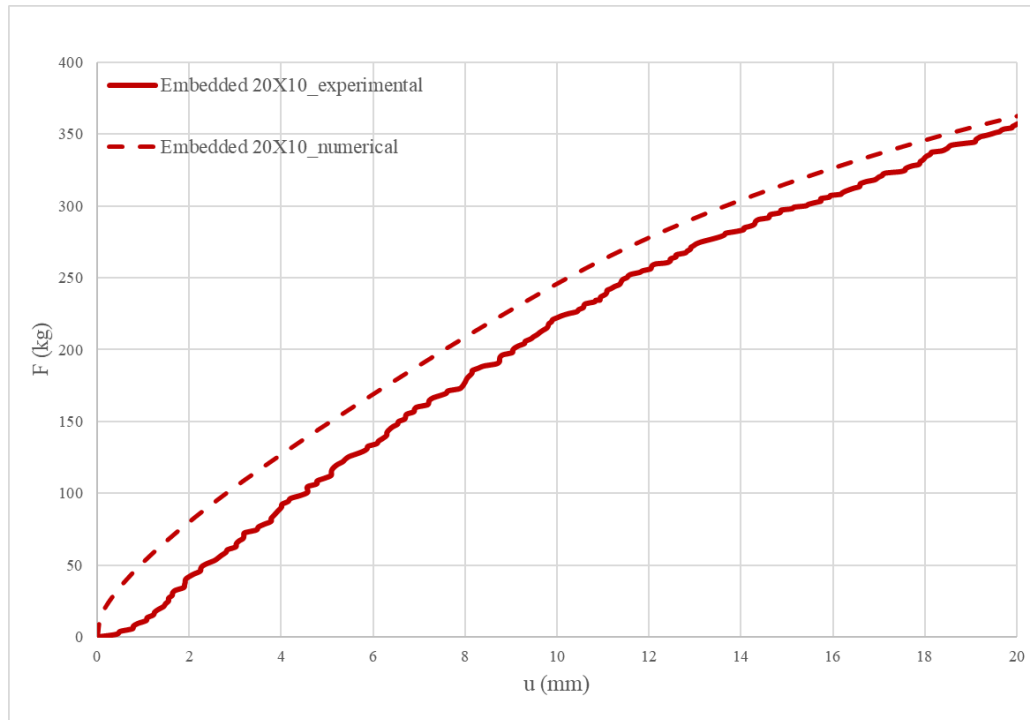


Figure 5.5. Load-settlement behavior of Embedded Foundation 20x10.

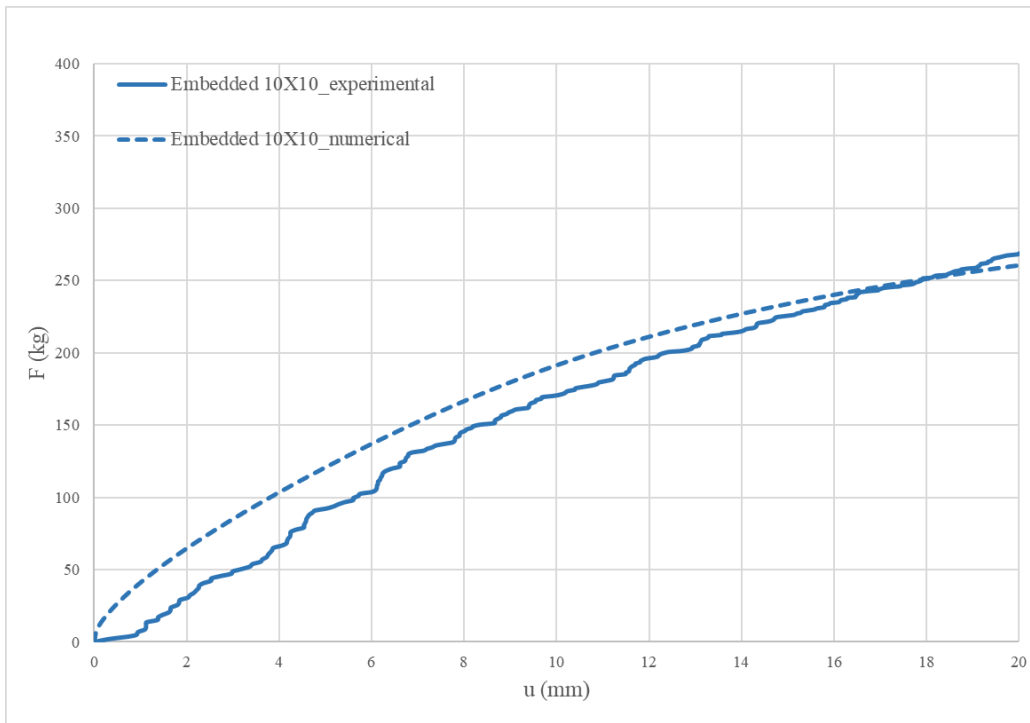


Figure 5.6. Load-settlement behavior of Embedded Foundation 10x10.

5.3. Monolithic Behavior of BSDFs

As previously explained, the term MBR refers to the “monolithic bearing ratio” which represents the ratio of the bearing capacity of a BSDF to that of a representative embedded foundation. The MBR value indicates the extent to which a BSDF functions as a compound body, with values closer to one indicating a more monolithic behavior.

Figure 5.7, 5.8, 5.9 and 5.10 present the load-settlement plots obtained through numerical simulations and experimental studies for both BSDFs and their embedded counterparts. The bearing capacity for all cases was calculated as 10% of the foundation width, equivalent to 10 mm.

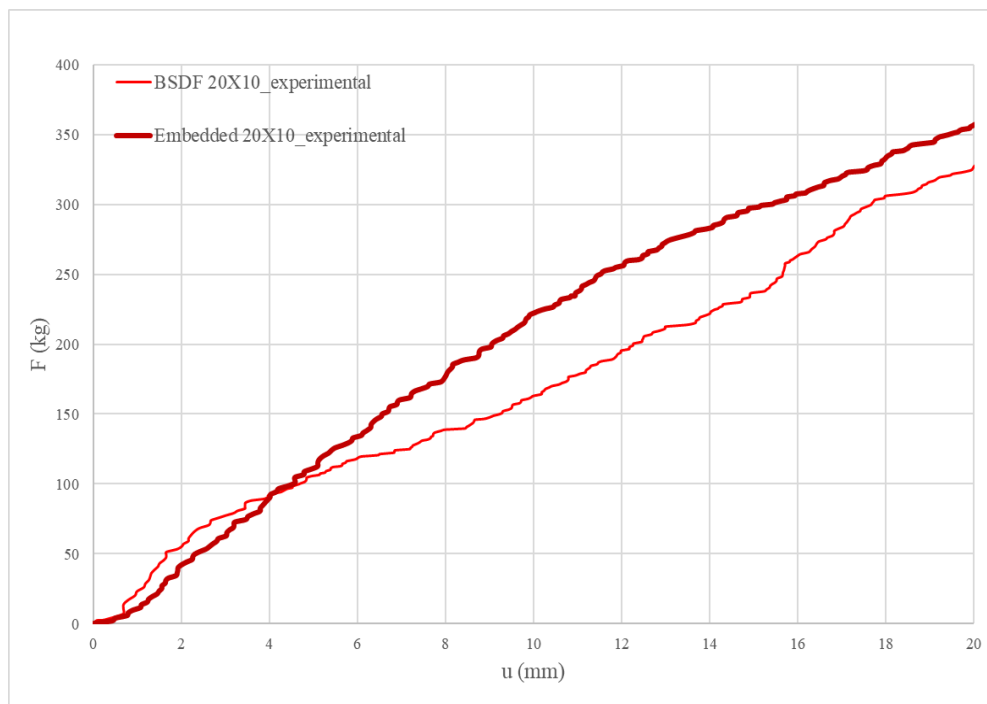


Figure 5.7. Load-settlement behavior of experimental studies with 20x10 foundations.

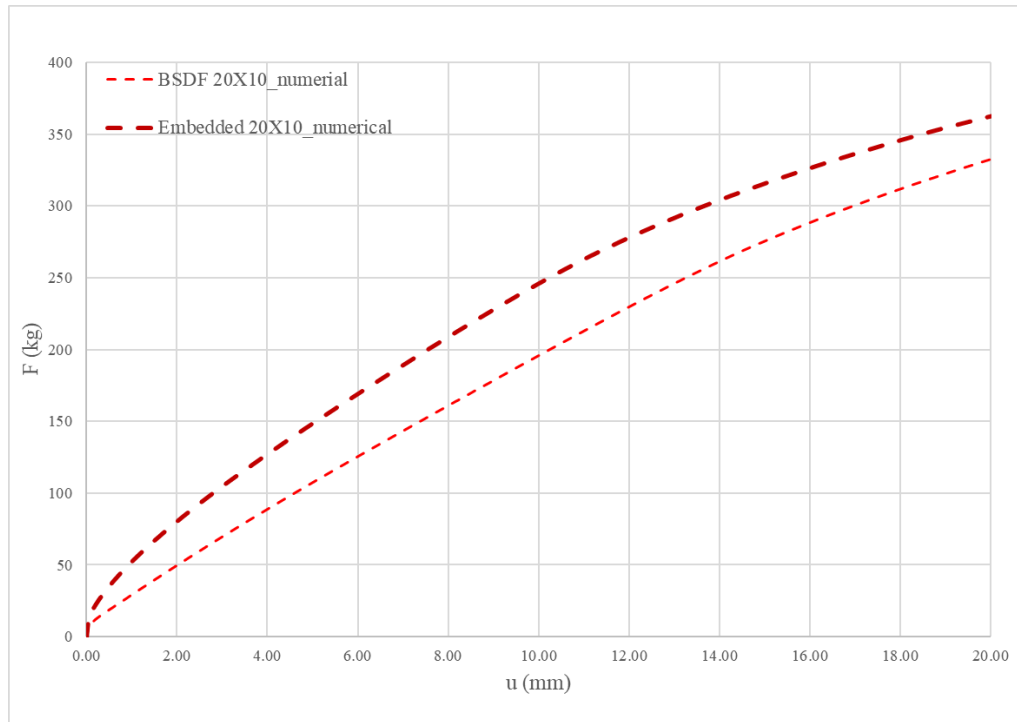


Figure 5.8. Load-settlement behavior of numerical studies with 20x10 foundations.

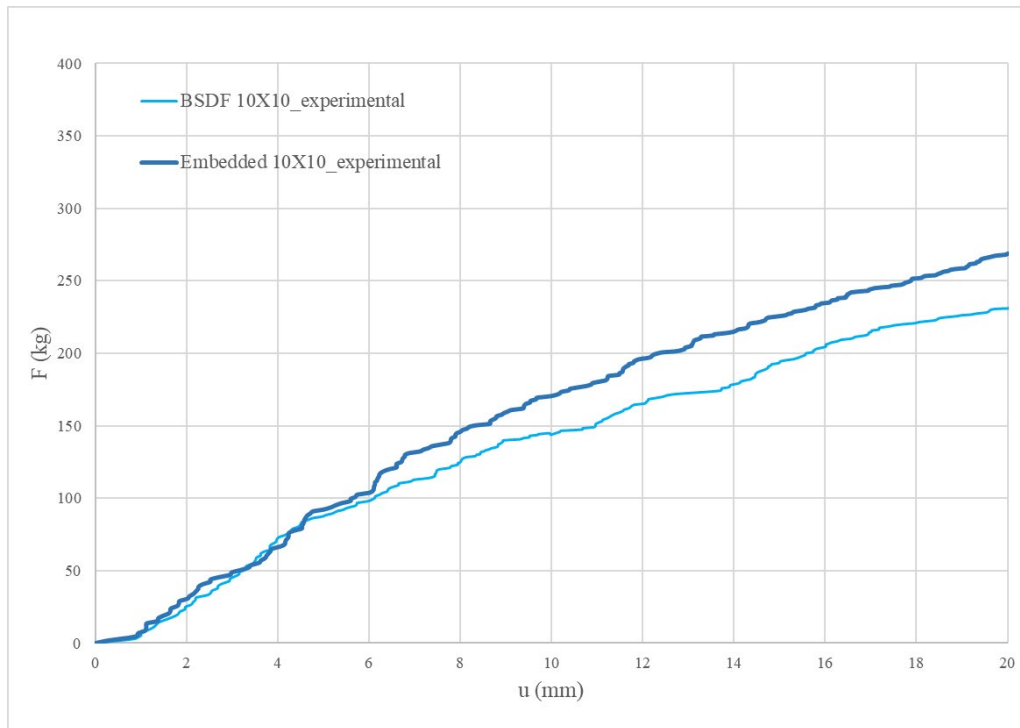


Figure 5.9. Load-settlement behavior of experimental studies with 10x10 foundations.

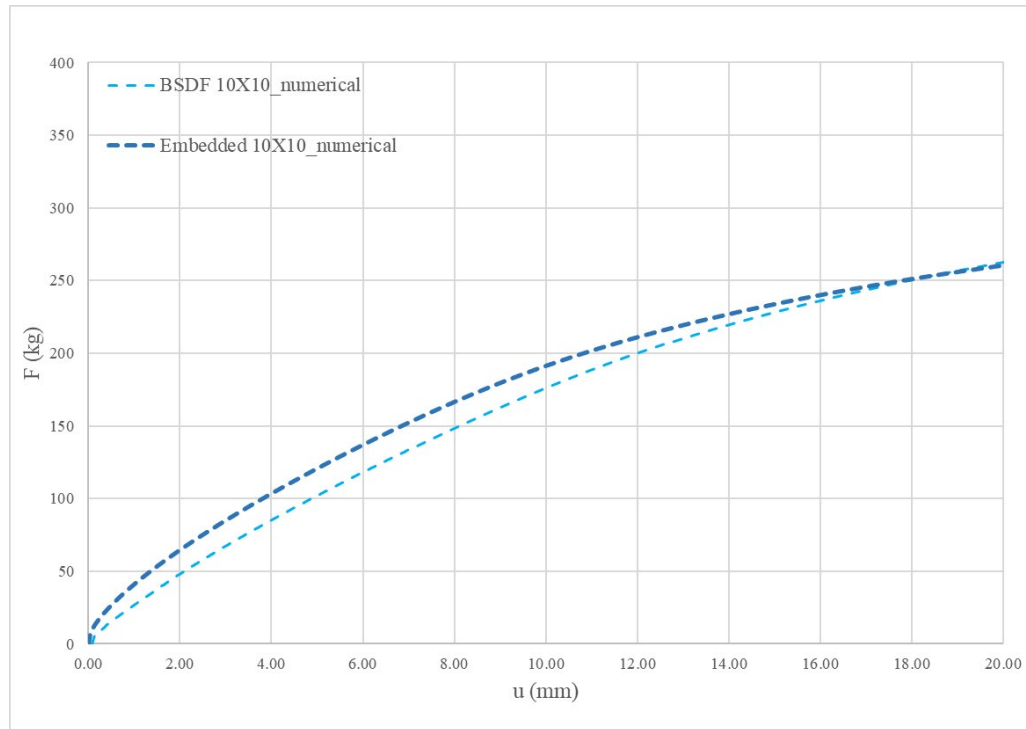


Figure 5.10. Load-settlement behavior of numerical studies with 10x10 foundations.

Table 5.2 presents the experimentally and numerically obtained load capacity values for BSDFs and their embedded counterparts. MBR values are also given as indicators of monolithic behavior. BSDF with 20 m depth and 10 m width, have MBR values which are 0.74 and 0.80 for experimental and numerical studies, respectively. As for BSDF with 10 m depth and 10 m width, MBRs increases to 0.86 and 0.92, respectively for experimental and numerical studies.

Table 5.2. Experimentally and numerically obtained MBR values.

Model Foundation	D/B	Load Capacity (kg)		MBR	
		Experimental	Numerical	Experimental	Numerical
BSDF 20x10	2	164.18	196.07	0.74	0.80
Embedded 20x10	2	222.34	246.16		
BSDF 10x10	1	146.14	176.51	0.86	0.92
Embedded 10x10	1	170.4	191.54		

6. DISCUSSION

In this section, assumptions for the models developed in numerical and experimental analyses will be discussed, and findings obtained from the analyses will be examined.

6.1. Performance Requirements

Leonards (1982) defined failure as “an unacceptable difference between expected and observed performance”. It directly mandates the engineer to assess the anticipated performance or performance prerequisites of a structure. Performance requirements for structures are primarily outlined as assessments of ultimate and serviceability limit states, along with considerations of constructability and economic requirements. Within the scope of this study, as described above, all four criteria have been taken into consideration for the selection of proper BSDF and CPRF configurations and the comparisons.

Bearing capacity within the scope of the ultimate limit state can be described as a peak point; beyond this point, an increase in load will result in a significant increase in foundation settlement. In an earlier work, Vesic (1963) conducted a study concerning the factors that impact the load-bearing capacity of deep foundations on sand. Subsequent large-scale model experiments have demonstrated that the failure mechanisms of foundations are influenced by both the compressibility of the sand and aspect ratio of the foundation. These foundation failures manifest in three distinct modes: general shear failure, local shear failure, and punching shear failure. Foundations on loose to medium dense sands and deep foundations with the aspect ratios higher than two succumb to the local or punching shear failure modes. Furthermore, it is important to note that general shear failures exhibit distinct failure points, unlike local and punching shear failures. As a result, defining a failure point for deep foundations becomes somewhat ambiguous.

Vesic (1977) has shown that many shallow foundations fail at the foundation settlement of four to 10% of B . This condition occurs concurrently with the general shear failure in sand. However, in the case of loose to medium sand and deep foundations, the ultimate load (V_{ult}) has the potential to reach a higher ratio, ranging from 15% up to 25%. In

this study, within the scope of ultimate limit state assessment, a stress at a settlement of 10% of B has been adopted for the determination of bearing capacity. On the other hand, BCR, SRR, MBR, and MSR coefficients have been determined using this ultimate limit state.

Secondly, serviceability limit state has been utilized for determining the performance of BSDFs and comparing them to CPRFs. The critical performance requirement for many foundations are often the total settlement and differential settlement rather than the ultimate bearing capacity. Structures encountering significant total settlements may encounter certain issues such as: distorted utility lines, connection and access problems with the existing structures and functionality problems for bridges. However, in the literature, there are different limit values for uniform and differential settlements that structures can tolerate. Coduto et al. (2016) stated that, certain structures have continued to operate without significant issues despite experiencing remarkably extensive total settlements; for instance, numerous buildings have shown minimal to no adverse consequences, even after undergoing settlements of up to 250 mm. Poulos (2017) reported that the acceptable limit for settlements was determined to be 106 mm, based on an analysis of 52 cases involving deep foundations, with a standard deviation of 55 mm. In addition, Coduto et al. (2016) provided permissible ranges for total settlements as 12 – 50 mm, 25 – 75 mm, and 50 mm for office buildings, heavy industrial buildings, and bridges, respectively. On the other hand, differential settlements tend to be more problematic than total settlements as they introduce structural distortions. Poulos (2017) recommended the accepted inclination limit for the majority of public buildings to be set at 1/300, to ensure proper functioning of elevators. It is known that rigid raft foundations are less sensitive to differential settlements.

In this study, the load-carrying capacity has been calculated considering a foundation settlement of 50 mm, which is in line with the acceptable serviceability limit. Differential settlements have not been considered as a comparison criterion within the scope of this study.

Cost-effective design is also a part of good engineering. Our aim should be to create designs that are both safe and efficient in terms of cost. In this context, a need arose for a parameter to compare BSDFs and CPRFs within the scope of cost-effectiveness. Furthermore, CPRFs exhibit variations in pile positions, pile numbers, pile lengths, raft-soil stiffness ratios, and load distribution on the raft. As a consequence, an extensive range of

configurations for CPRFs can be established, allowing for comparison with BSDFs. To enable a purposeful comparison between BSDFs and CPRFs, a key parameter, namely the concrete volume of the deep foundations, has been taken into account. Specifically, deep foundations possessing similar or identical concrete volumes have been subject to evaluation based on the efficiency concept.

Lastly, constructability has also been taken into consideration as a comparison criterion for the selection of proper BSDF and CPRF configurations. As mentioned in the Section 1, to achieve an enclosed structure around the circumference of BSDFs, options such as sheet piles, diaphragm walls (barrette foundations), or drilled piles may be selected. Similar to rotary bored piles, barrettes are created using drilling methods while utilizing bentonite fluid or polymers to maintain bore stability during the drilling process. Unlike bored piles, barrettes are not inserted using rotary drilling rigs; instead, they are formed using a diaphragm wall cutter, such as a hydrofraise, or through the utilization of a clamshell. According to Poulos (2017), barrettes hold an advantage over conventional bored piles due to their ability to be installed to depths exceeding 100 m, providing a heightened level of vertical precision and the capability to withstand larger loads. Taking this situation into consideration, the maximum depths are selected as 96 m and 60 m for BSDFs and CPRFs, respectively.

6.2. Load-settlement Behavior

As a rule, utilizing BSDFs can lead to an improvement in bearing capacity and a reduction in settlements. Nonetheless, it should be noted that the load-settlement behavior depends on various factors, including the geometry of BSDFs, soil conditions, surface roughness, load level, and load configuration. In Section 4.1, the mentioned factors and their impacts have been investigated in detail. Here, the mechanism behind the encountered situations will be investigated and discussed with the assistance of soil mechanics and previous studies.

As the depth of a BSDF increases, it is expected to have a greater bearing capacity and less settlement compared to a shallow foundation with the same width. Therefore, as the depth increases, the BCR increases while the SRR decreases. On the other hand, for BSDFs

at equal depths, as the width increases, they tend to exhibit a smaller increase in bearing capacity and a smaller reduction in settlement compared to shallow foundations with the same width. In other words, increasing the width is associated with a lower BCR and a higher SRR tendency. The increase in the wall-to-raft volume ratio, as demonstrated in Table 6.1, can explain the greater bearing capacity and reduced settlements observed in BSDFs with smaller widths but the same depth when compared to their counterpart shallow foundations. This higher ratio enables smaller-width BSDFs to have a larger area for acquiring greater skin resistance compared to the raft area of counterpart shallow foundations.

As the aspect ratio increases, there is a tendency for BCR values to rise and SRR values to decline. However, it's important to recognize that various geometries sharing the same aspect ratios may exhibit different BCR and SRR values. Table 6.1 and 6.2 illustrate the wall-to-raft volume ratios with the changing widths and depths, respectively. Increasing aspect ratio values tend to increase the wall-to-raft volume ratio and, consequently, enhance BCR while reducing SRR.

Table 6.1. Wall-to-raft volume ratio with the increasing width.

D (m)	B (m)	D/B	V_r (m³)	V_w (m³)	V_w / V_r	BCR	SRR
20	4	5.00	32	65.28	2.04	4.42	0.04
20	8	2.50	128	142.08	1.11	2.44	0.22
20	12	1.67	288	218.88	0.76	1.88	0.43
20	16	1.25	512	295.68	0.58	1.55	0.58
20	20	1.00	800	372.48	0.47	1.35	0.70
20	30	0.67	1800	564.48	0.31	1.16	0.85
20	40	0.50	3200	756.48	0.24	1.13	0.88

Table 6.2. Wall-to-raft volume ratio with the increasing depth.

D (m)	B (m)	D/B	V_r (m³)	V_w (m³)	V_w / V_r	BCR	SRR
4	8	0.50	128	71.04	0.56	1.42	0.68
8	8	1.00	128	142.08	1.11	1.58	0.60
12	8	1.50	128	213.12	1.67	1.79	0.50
16	8	2.00	128	284.16	2.22	2.07	0.35
20	8	2.50	128	355.2	2.78	2.44	0.22
30	8	3.75	128	532.8	4.16	3.68	0.07
40	8	5.00	128	710.4	5.55	4.97	0.04

In this study, BCR and SRR values were obtained for three distinct types of sand: S_1 , S_2 , and S_3 . As anticipated, denser soils exhibit greater bearing capacities across all aspect ratios. Nevertheless, it's worth noting that, regardless of depth, width and aspect ratio, the increase in BCR is more pronounced in loose sand compared to medium and dense sands. There is a noticeable correspondence between SRR and BCR values, indicating that the inclination for settlement reduction is more pronounced in sands with lower relative density.

Comparing the BCR and SRR values obtained in this study with those reported in previous literature reveals a consistent trend across different aspect ratios. Moreover, it's evident that skirted foundations excel in enhancing bearing capacity and reducing settlement, particularly in low relative density sands, as opposed to medium or high relative density sands, as cited in Section 2 (Villalobos, 2007; Khatri et al., 2017; Sajjad and Masoud, 2018). Following experimental tests conducted on skirted foundations up to an aspect ratio of two, it has been determined that this phenomenon can be attributed to the greater potential for enhancing the shear strength of loose soils compared to dense soils. In other words, the BCR is higher in loose soil because the required displacement to mobilize strength in dense soil is insufficient compared to that in loose soil. This situation is also consistent with the tendency for the failure mode to approach general shear failure as the relative density increases.

To account for the influence of foundation surface roughness in the models, a strength reduction factor has been employed to establish a connection between interface strength and soil strength. Three distinct R_{int} values, specifically 0.6, 0.8, and one, have been chosen to depict the interaction between soil and structure. As R_{int} rises, both bearing capacity and BCR experience an increase. Moreover, the findings indicate that as the surface roughness of foundation models increases, the enhancement in bearing capacity values becomes more pronounced. Conversely, settlements and SRR values decrease with higher R_{int} values. This observation could be linked to the BSDF walls, which, as the surface roughness decreases, appear to provide insufficient assistance in load transfer. Consequently, a noteworthy portion of the uniaxial loads is directed towards the tip level, causing a decline in the bearing capacity and BCR.

When interpreting the impact of the load level on load-settlement behavior of BSDFs, it is beneficial to explain how the BCR values have been obtained. As explained in Section 4.1, BCR can be defined as the ratio of bearing capacity of BSDFs to the bearing capacity of representative shallow foundations at a load level resulting in a settlement equal to 10% of the B . However, to accommodate different load scenarios, BCR values have also been established through the assessment of bearing pressures at settlements corresponding to 1.25%, 2.5%, and 5% of the B . As a result, until the aspect ratio of one, BCR values are higher for higher load levels, though BCR values tend to decrease as load levels increase after the aspect ratio of one. In other words, lower aspect ratios tend to result in higher BCR values for higher load levels, whereas higher aspect ratios show higher BCR values at lower load levels. The reason might be that, at lower load levels, the load is mostly supported by the shaft resistance since a significant portion of the base resistance has not yet been fully activated. As mentioned above, wall-to-raft volume ratio is higher for higher aspect ratios which implies an increase in shaft resistance. So, as a result, BCR values tend to increase more steeply for lower load levels at higher aspect ratios.

Following the experimental load-settlement tests on the model foundations outlined in Section 5.2, numerical analyses were conducted using Plaxis 3D to validate the test results and draw conclusions regarding the performance of parametric studies. Comparing the numerical and experimental simulations, the load capacity difference between BSDFs with dimensions of 20 cm in depth and 10 cm in width is 16%. For the embedded counterparts of BSDF 20x10, this difference reduces to 10%. In the case of BSDFs with dimensions of 10 cm in depth and 10 cm in width, the load capacity difference is 18%. Meanwhile, the embedded counterparts of BSDF 10x10 exhibit a load capacity difference of 11%.

Physical model tests were conducted on BSDFs and their embedded foundation counterparts to confirm the validity of the numerical approach outlined in this study, and the results indicated a satisfactory agreement considering the load-settlement behavior.

6.3. Monolithic Behavior

The main objective of a BSDF is to create a soil volume within its walls and establish a firm core that serves as an integral load transfer component. Nevertheless, the

incorporation of this soil core introduces complexities into the behavior of BSDFs. To tackle this challenge, two approaches have been suggested to assess the extent to which a BSDF operates as a composite element. The first approach, known as the “single element approach”, considers shaft friction along both the external and internal surfaces of the walls. It also independently calculates the base resistance of the walls and the bearing capacity of the raft. The second approach, referred to as the “monolithic approach”, considers the full composite effect between the soil core and the structural elements. It takes into consideration the skin friction along the external surface of the box and the bearing capacity of an equivalent shallow foundation at the base to determine the overall bearing capacity of the BSDF. Consequently, the actual behavior of the BSDFs falls within the spectrum defined by these two approaches. Within the framework of this study, the quasi-monolithic behavior of BSDFs was examined through a comparison with embedded foundations, assuming that embedded foundations demonstrate complete monolithic behavior.

It is obvious that the complexity of BSDF behavior is influenced by interactions between the box wall and raft, the box walls, the raft and soil core, the box wall and soil core, as well as shaft-base resistances. Nevertheless, an analytical calculation of the load capacity of a BSDF will be performed for comparison with the numerical simulation results.

As mentioned above, using the single element approach and the monolithic approach, the total load capacities of BSDFS can be calculated separately using the total load capacity calculation method for single piles given in Section 2.2. The total capacity of a single pile in sand is typically estimated as $P_u = q_b A_b + \tau_s A_s$, where P_u is ultimate capacity, q_b and τ_s are base and shaft resistances, and A_b and A_s are areas of pile base and shaft, respectively. Base and shaft resistances can be calculated as $q_b = \sigma'_z N_q^*$ and $\tau_s = K \tan \delta \sigma'_z$, respectively. Different from the bearing capacity factors in shallow foundations, N_q^* used in deep foundations depends on both compressibility and shear strength of the soil. δ and K are soil-foundation interface friction angle, and earth pressure coefficient on the shaft, respectively.

A representative model with dimensions of $D=20$ m, $B=8$ m, $t=0.8$ m, and $T=2$ m, and lying on soil S_1 , whose engineering parameters were presented in Table 4.1 was selected for the analytical calculation. δ and K values have been calculated based on the studies by Kulhawy (1984) and Reese and O'Neil (1989), and considering the $R_{int}=0.6$ in the numerical

analysis. N_q^* used in analytical calculation has been chosen between 15 and 25, considering the studies of Coduto et al. (2016) and Canadian Foundation Engineering Manual (2006). For the analytical calculation, the range of load capacity values has been calculated as 3169-4350 kN/m² and 5186-8466 kN/m², respectively, considering the single element approach and the monolithic approach. In numerical calculations, the load capacity values calculated for the same representative model of BSDF and embedded foundation are 3456 kN/m² and 4640 kN/m², respectively. To sum up, as previously mentioned, the actual behavior of the BSDFs falls within the range defined by the single element approach and the monolithic approach. The values obtained from the analytical calculations are larger than the values from the numerical analysis, but they are in close proximity to each other.

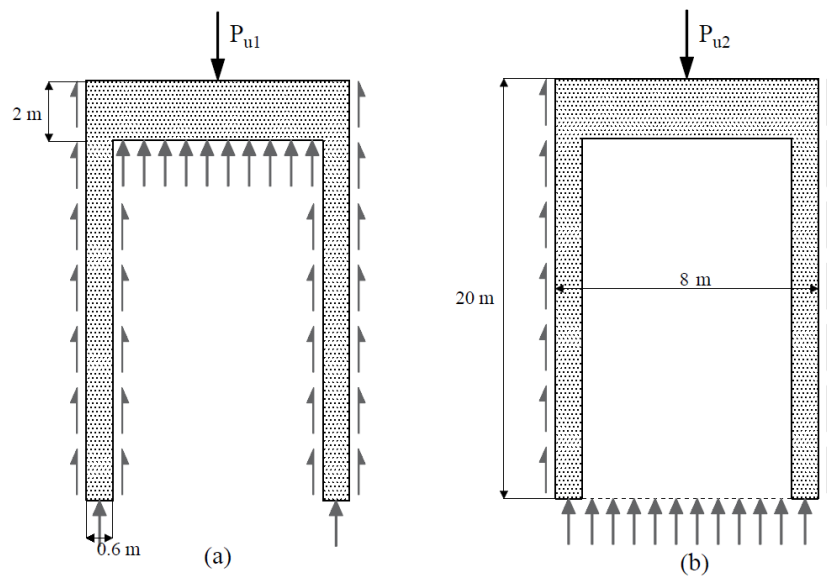


Figure 6.1. Single element approach (a) and monolithic approach (b).

As mentioned in Section 4.2, the MBR and MSR values serve as indicators to determine the degree to which a BSDF behaves like a compound body. The closer the MBR and MSR values get to one, the more a BSDF behaves like a monolithic compound body.

The geometry of BSDFs plays a significant role in monolithic behavior of BSDFs. Regarding the depth, initially, MBR values move away from one with increasing depth, but after reaching a depth of 20 m, this pattern shifts, and MBR values start to exhibit a slight increase which is a maximum of a two percent. This slight increase in MBR might be

attributed to a gradually decreasing gradient of end-bearing pressure with depth. Namely, the increase in bearing capacity of an embedded foundation decreases compared to the counterpart BSDF after a certain depth. Vesic (1969), Kulhawy (1984) and Randolph (1994) indicated that the average shaft friction and base resistance did not exhibit a linear increase with depth, instead, they reached their maximum values at a depth depending on the relative density of the soils. There is also a predominant perspective suggesting that shaft friction and base resistance may indeed continue to rise with greater depth, but at a diminishing rate (Fellenius, 2023). This phenomenon was attributed by Poulos (2017) to a combination of arching, the influence of soil compressibility, and the decrease in friction angle as the stress level increases.

MSR values experience a substantial increase and moves away from one as depth initially increases, but they level off after reaching a depth of 12 meters. Providing that MSR values are calculated as the settlement ratio between a BSDF and an embedded foundation, both subjected to the bearing pressure of a representative BSDF, a MSR behavior similar to the MBR behavior that varies with depth, as mentioned above, is observed.

As the width increases, MBR values at the equal depths tend to move away from one, but after reaching a width of 12 meters, this pattern shifts, MBR values start to exhibit a significant increase and approach unity. This behavior can be attributed to BSDFs behaving more monolithically for aspect ratios lower than 1.5, resembling shallow foundations more as the width increases. On the other hand, MSR values initially increase consistently and approach unity as width increases.

Generally speaking, BSDFs with lower aspect ratios tend to exhibit MBR and MSR values closer to one, behaving more like a monolithic-compound body, in contrast to BSDFs with higher aspect ratios. Experimental study in Section 5.3 also justified that as aspect ratio decreases, the MBR approaches one and monolithic behavior increases.

The impact of surface roughness on MBR was examined through multiple simulations involving different R_{int} values, namely 0.6, 0.8, and one. BSDFs having rough interfaces tend to display MBR and MSR values that are closer to one, behaving in a manner more akin to a monolithic-compound body, as opposed to BSDFs with smoother interfaces. On the

other hand, with an increase in R_{int} values, the influence of depth on both MBR and MSR values diminishes, leading to reduced fluctuations in these values. Assuming a rough interface between the soil inside the BSDF and the wall ensures that there is no slippage happening at the interface. This is because the relative deformation of the soil enclosed within the walls is limited, causing both the soil and the walls to move together in unison.

To sum up, the extent of soil compression in the box decreases with depth and increases with width while increasing with soil roughness. This is also an indicator of closeness to monolithic behavior. In other words, BSDFs with soil cores getting compressed more act much more like embedded foundations. Figure 6.2 and 6.3 show the displacement profiles of BSDFs for different aspect ratios and levels of surface roughness, respectively.

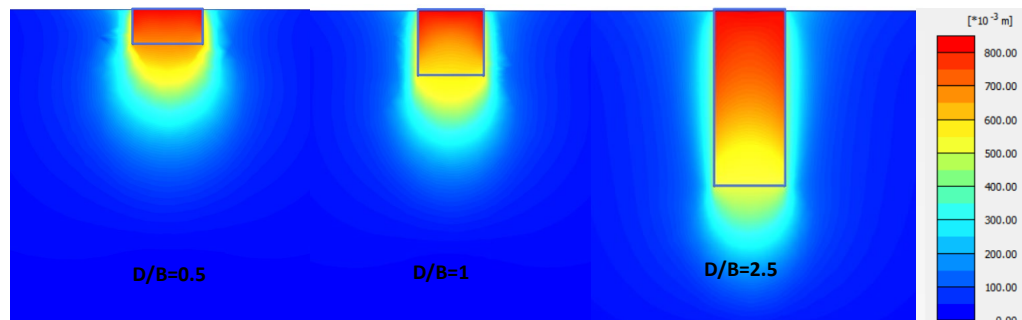


Figure 6.2. Displacement profiles for different aspect ratios.

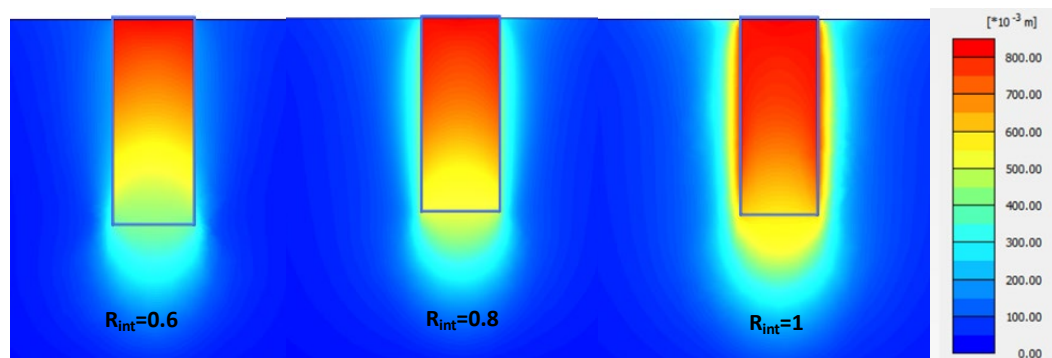


Figure 6.3. Displacement profiles for different levels of surface roughness.

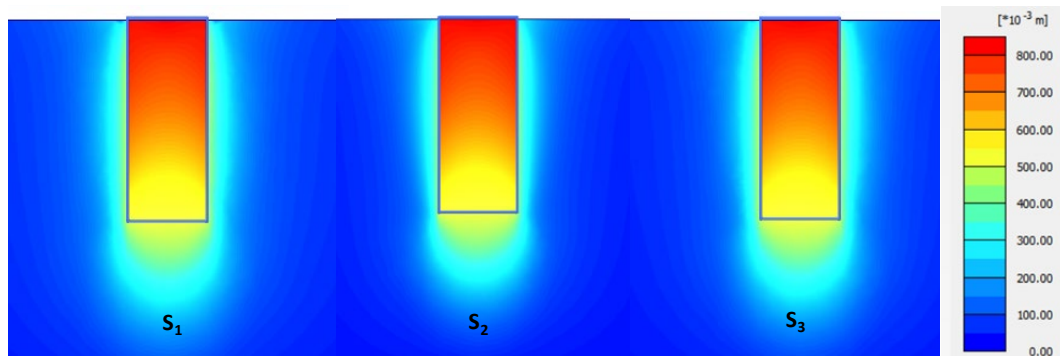


Figure 6.4. Displacement profiles for different soil types.

Although it may not be as apparent as the load-settlement behavior, the monolithic behavior of BSDFs is also influenced by the relative density of the underlying sand. Denser sands exhibit more monolithic behavior, particularly when considering lower aspect ratios. This can be attributed to the enhanced efficiency of arching in denser soil (Terzaghi, 1943). Figure 6.4 shows the displacement profiles of BSDFs for different soil types.

Load level is one of the factors having an impact on monolithic behavior of BSDFs. As detailed in Section 4.2, MBR can be defined as the ratio of the bearing capacity of BSDFs to that of counterpart embedded foundations at a load level resulting in a settlement equal to 10% of B . However, to accommodate various load scenarios, MBR values have also been determined by assessing bearing pressures at settlements corresponding to 1.25%, 2.5%, and 5% of B . Reduced aspect ratios typically result in MBR values approaching unity as load levels increase, whereas higher aspect ratios lead to MBR values approaching unity at lower load levels. In essence, monolithic behavior intensifies as load levels rise for smaller aspect ratios, while it diminishes as load levels increase for larger aspect ratios. This behavior might be attributed to the fact that, at lower load levels, a significant portion of the load is predominantly supported by shaft resistance since much of the base resistance has not yet been fully engaged. As mentioned earlier, higher aspect ratios lead to a greater wall-to-raft volume ratio, which results in an increased shaft resistance. Therefore, MBR values tend to approach unity more during lower load levels for higher aspect ratios.

At this point, it is worthwhile to discuss soil arching, which is one of the factors influencing monolithic behavior. As the foundation and the surrounding soil settle downward, the soil situated at the lower portion of the foundation experiences greater

confinement compared to the soil near the lower part of the soil core, giving rise to the phenomenon known as soil arching. In other words, the differences in settlement between the foundation walls and the adjacent soil develop at the foundation base, giving rise to soil arching.

Terzaghi (1943) characterized soil arching as the process by which pressure is transmitted from a yielding soil mass to adjacent stable elements. This phenomenon was effectively demonstrated in a renowned experiment conducted by him. In the experiment, he measured the force required to support a trap door positioned at the bottom of a box filled with sand. The measurements illustrated a significant decrease in the force exerted on the trap door as it descended away from the box. Terzaghi attributed this reduction to the arching effect, wherein the interlocking particles of the soil redistributed stress around the opening. Terzaghi (1943) noted that the magnitude of this force reduction was much more pronounced when dealing with dense sand in comparison to loose sand. This emphasized the enhanced effectiveness of arching in denser soil. Furthermore, it was observed that arching manifested at smaller displacements in the denser soil as opposed to the loose sand. As the trap door continued to descend, the arching eventually collapsed, and the force exerted on the door gradually returned to levels resembling the original values.

Paikowsky (1989) demonstrated that the redistribution of stresses within a soil mass involves the phenomenon of arching, particularly in cohesionless soils. In the context of installing open-ended piles, there is relative movement between the pile walls and the surrounding soil mass. Paikowsky (1989) referred to this as passive arching, distinguishing it from the active arching concept found in soil between retaining walls and backfill of trench excavation. In the case of active soil arching, the soil settles relative to the walls, whereas in passive arching, the walls either move downward in relation to the soil or the soil is pushed upward. When passive arching occurs around an open-ended pile, it results in the formation of a convex shape in the soil near the pile base. In this scenario, the arching mechanism effectively transfers the axial stress acting on the internal soil column at the pile base to the pile walls in the form of horizontal stress, leading to an increase in internal shaft friction.

Arching characterizes the curved and continuous distribution of principal stresses. Effective principal stress directions shows that soil arching occurs on the bottom of the BSDFs.

Arching leads to stress transmission from a yielding soil mass to adjacent stable elements. This phenomenon might reduce the settlement of BSDFs. Furthermore, arching results in a rotation of the principal plane towards the walls, leading to an increase in horizontal stress and shaft friction on both inner and outer walls of BSDFs. Terzaghi (1943) showed that the arching is more efficient in denser soils. Furthermore, McNulty (1965) conducted a trap door experiment which was similar to the experiment of Terzaghi (1943). The experiment of McNulty (1965) demonstrated that passive arching exhibits a more pronounced increase in response to increases in both the aspect ratio and the relative density of sands.

Based on these studies mentioned above, it can be deduced that BSDFs with higher aspect ratios and relative densities might show more pronounced arching effect compared to BSDFs with lower aspect ratios and relative densities. Nevertheless, it's important to keep in mind that monolithic behavior is a function of factors like arching and wall-to-raft volume ratio as well as the degree of soil compression based on geometry and surface roughness.

6.4. Comparison of BSDFs and CPRFs

Reul and Randolph (2004) conducted simulations involving different CPRFs to examine their load-settlement characteristics. These parametric investigations revealed that achieving an optimal design for a CPRF depends on several factors, including the underlying soil conditions, load level and configuration, pile arrangement and foundation geometry. In the context of CPRFs, one can vary parameters such as pile configuration, the number of piles, pile length, the ratio of raft-to-soil stiffness, and how the load is distributed across the raft.

Similar to CPRFs, BSDFs also exhibit different geometries, which can vary in terms of box lengths, widths, and numbers, as well as different wall and raft thicknesses. Furthermore, load levels, load configurations, and soil types can also vary, and these factors

play a prominent role in altering the behavior of BSDFs. Consequently, there exists infinite number of BSDF configurations that can be compared with CPRFs.

In order to enable a meaningful comparison between BSDFs and CPRFs, we have taken into account a crucial factor: the concrete volume in the deep foundations. More precisely, we will evaluate deep foundations with similar or identical concrete volumes using the efficiency concept. As mentioned in Section 4.3, efficiency refers to a state in which a system or machine operates optimally. The variable $P\%$ represents this “efficiency”, signifying the percentage change in load capacity of BSDFs compared to CPRFs with equivalent volumes.

The load capacity has been calculated based on a foundation settlement of 50 mm, in accordance with the serviceability limit states among the performance requirements mentioned in Section 6.1. In other words, a positive value of $P\%$ indicates that BSDF has a greater load capacity than CPRF among the compared deep foundations when subjected to a 50 mm settlement.

Fellenius (2023) highlighted that calculating the raft thickness is an integral aspect of the design process, and it is primarily influenced by the pile diameter and the spacing between piles. An increase in pile spacing results in a corresponding increase in raft thickness. It's important to note that closer spacing can potentially lead to an overlap of the stress transmitted by the piles to the soil, which, in turn, reduces the load capacity of the pile group and increases the group settlement. Therefore, for CPRFs, a recommended pile spacing of 2.5-three times the pile diameter has been chosen for the optimal performance, and in this case, the pile spacing has been set at three times the pile diameter. The pile diameter and box thickness have been set at one and 0.6 meters, respectively. In order to reduce the differential settlements, a two-meter raft thickness has been opted for, creating a rigid raft. The surface between the sand and concrete was considered to be rough, thus, there was no reduction in resistance attributed to the interface between them.

In the scope of this study, deep foundations with the same volume and raft widths were initially compared, followed by a comparison of deep foundations with the same volume but different raft widths. The comparisons made while keeping the total volume constant involve

deep foundations with the same width, which were designed to represent building-type structural foundations, while those with different widths were designed to represent bridge-type structural foundations. In other words, the foundations of building-type structures encourage the construction of foundations that encompass the largest area allowed by the parcel boundary, thus leading to the selection of the widest possible foundation width when box wall widths can be reduced. In this case, comparisons were made by selecting CPRF and BSDF raft widths to be equal by changing the box widths and keeping the deep foundation volume constant. The Section 4.3.2 covers the deep foundations with the same total volumes and raft widths.

When it comes to bridge-type structural foundations, reducing the construction area, while considering the dimensions of the carried bridge pier, is also possible. Reducing the foundation construction area for bridges has the advantage of keeping more lanes open for traffic during the construction of the bridge. In areas with reduced raft width, pile spacing may significantly decrease in the case of CPRFs and may not be favored, therefore, BSDFs may provide an alternative to CPRFs. The Section 4.3.3 covers the deep foundations with the same total volumes but different raft widths.

In Section 4.3.2, CPRF configurations with four piles, 25 piles, and 49 piles were compared to BSDFs with equivalent volumes in order to evaluate their efficiency. This analysis involved a total of 180 unique models, spanning a range of CPRF aspect ratios from one to four. For each model, simulations were conducted using three different soil types, facilitating additional opportunities for comparison and interpretation. It's essential to emphasize that within Section 4.3.2, the deep foundations being compared share identical raft volumes and geometries, with adjustments made to the wall or pile volumes to ensure an equivalent total volume.

Through the comparison of BSDFs and CPRFs with identical volumes, the parametric study revealed that the efficiency of BSDFs is predominantly influenced by their geometry. As a result, making a broad generalization about the efficiency of BSDFs in all practical scenarios is unfeasible. Nonetheless, the essential outcomes of this parametric study can offer valuable guidance during the initial stages of a project when deciding which deep foundation configuration options to explore.

Figure 4.31, 4.32, 4.33 and 4.34 illustrates that among all deep foundation configurations, as the raft width increases, compared deep foundations tend to exhibit positive $P\%$ values which means positive efficiency. Namely, achieving a positive $P\%$ implies that, considering the compared deep foundations, BSDF outperforms CPRF by exhibiting a greater load capacity in response to a 50 mm settlement. Furthermore, as the aspect ratio of counterpart CPRFs increase, $P\%$ tends to vary within a larger range of values. For all deep foundation configurations, after an aspect ratio which varies but is approximately one, as the aspect ratio of a BSDF increases, so does the $P\%$ value. For a specific aspect ratio, BSDFs tend to exhibit slightly higher efficiency values over denser soil for widths of 5 and 14 meters. Nevertheless, BSDFs with a 20-meter width tend to have slightly higher efficiency values in the case of relatively loose sands. In other words, relative density does not play a significant role in efficiency.

No common findings for compared deep foundations have been encountered for all configurations beyond the points mentioned above. It is certain that BSDFs are not superior to CPRFs in all cases or vice versa. However, positive $P\%$ values are predominantly observed in larger-width rafts, where BSDF aspect ratios are generally higher than CPRF aspect ratios. Furthermore, as the aspect ratio of a BSDF rises, the $P\%$ value also tend to increase.

Considering the load-settlement behavior of deep foundations, there are shaft and base resistances withstanding the applied load. Specific surface is defined as the ratio of the perimeter to the cross-sectional area of either a barrette or a circular pile. A rectangular barrette exhibits a greater specific surface area compared to a circular pile with an equivalent cross-section that relies solely on shaft resistance to resist vertical loads (Durgunoğlu, 2013). A similar ratio has been introduced as the “hydraulic radius” concept by Brandl (2001), defined as the ratio between box area and box circumference. The specific surface values for CPRF and BSDF configurations in Section 4.3.2 are four and 3.33, respectively. CPRFs exhibit a greater specific surface area and, as a result, shaft resistance. However, some BSDF configurations with higher aspect ratios exhibit positive $P\%$ values. These BSDFs with lower specific surface area compared to their CPRF counterparts might have compensated for this gap with tip resistance. Here, tip resistance actually refers to the monolithic behavior formed by the contribution of the tip portion of BSDF walls and the soil core. Even at relatively low

load levels corresponding to a settlement of 50 mm, it has been observed that monolithic behavior reaches levels of around 70-80% in Section 5.3.

In Section 4.3.3, the efficiency of BSDF configurations was evaluated by comparing them to CPRFs with identical volumes, each featuring nine piles. A total of 15 unique models were simulated, covering a range of BSDF aspect ratios from 0.81 to 4.68, while the counterpart CPRF had a fixed aspect ratio of one. For each aspect ratio, three different soil types were taken into account, allowing for further analysis and comparison. Unlike the deep foundation configurations in Section 4.3.2, it is worth emphasizing that within this section, the compared deep foundations have varying raft volumes and geometries, with the volume of walls or piles being altered to ensure an equal total volume. Furthermore, it should be considered that with changing foundation thicknesses, both the box and raft volumes will vary, consequently leading to variations in the $P\%$ values. Efficiency values experience a significant rise as the aspect ratios of BSDFs increase, beginning at -21.94% for an aspect ratio of 0.81 and reaching their peak at 73% for an aspect ratio of 4.68. Regarding the influence of soil relative density, BSDFs situated in less compacted soil exhibit higher $P\%$ values when compared to those in denser soil. This behavior becomes notably more pronounced beyond an aspect ratio of one. The reason behind the positive $P\%$ values could be the increasing BSDF aspect ratios, which lead to an increase in surface area, and also the contribution of the soil core due to monolithic behavior.

As mentioned above, reducing the foundation construction area for bridges offers the benefit of maintaining additional lanes for traffic accessibility throughout the bridge construction phase. Therefore, taking into account the functional benefits as well, BSDF configurations with positive $P\%$ values might be considered as superior to CPRFs.

7. CONCLUSIONS

The main objective of this research was to evaluate the load-settlement behavior of BSDFs in sand, subjected to static vertical loads. This study also seeks to expand the existing knowledge required to comprehend the monolithic characteristics of BSDFs. Furthermore, to assess the efficiency of BSDFs, a comparative analysis of the load-settlement behavior of BSDFs and CPRFs have been performed. Finally, physical model tests have been carried out to confirm the numerical method proposed in this study, and the results showed a reasonable level of agreement.

The impact of parameters such as depth, width, aspect ratio, surface roughness, and soil type, as well as load level, on behavior was comprehensively investigated through a parametric study.

Two coefficients, namely BCR and SRR, were formulated to provide a more comprehensive interpretation of the load-settlement characteristics of BSDFs. BCR can be defined as the ratio between the bearing capacity of BSDFs and the bearing capacity of counterpart shallow foundations at a load level that induces a settlement above the performance limits. For ultimate limit state requirement, this settlement is assumed to be equivalent to 10% of the foundation width. SRR is represented as the ratio of the settlement observed in a BSDF to the settlement experienced by a representative shallow foundation under constant bearing pressure, equivalent to the capacity of the representative shallow foundation with the same width. The capacity load has initially been defined as the load level that leads to a settlement equal to 10% of the foundation width. Nevertheless, to account for various load levels, BCR values might also be determined by examining bearing pressures at settlements corresponding to 1.25%, 2.5%, and 5% of the foundation width.

Reducing the depth while increasing the width is linked to a decreased BCR and an elevated SRR tendency. In other words, when the aspect ratio increases, BCR also increases, while SRR decreases. This can be attributed to the increasing wall-to-raft volume ratio as aspect ratio increases. This increasing ratio allows BSDFs to possess a larger surface and

base areas, which in turn enables them to attain higher skin and base resistances when compared to the raft area of equivalent shallow foundations.

Additionally, there is a greater inclination toward increased bearing capacity and reduced settlement in loose sand when compared to denser sands. In other words, as the relative density of the soil rises, BCR declines while SRR increases.

To incorporate the impact of foundation surface roughness into the models, a strength reduction factor has been utilized to establish a relationship between interface strength and soil strength. Three different R_{int} values, namely 0.6, 0.8, and one, were selected to illustrate the interaction between the soil and the structure. As R_{int} increases, BCR increases while SRR decreases.

To account for various loading levels, BCR values were also established by evaluating bearing pressures at settlements corresponding to 1.25%, 2.5%, and 5% of the foundation width. Consequently, up to an aspect ratio of one, BCR values exhibit a tendency to be higher for increased load levels, but they tend to decrease as load levels rise beyond an aspect ratio of one. In simpler terms, lower aspect ratios generally lead to higher BCR values when load levels are increased, whereas higher aspect ratios result in higher BCR values at lower load levels. This phenomenon may be attributed to the fact that at lower load levels, the majority of the load is primarily supported by shaft resistance since a significant portion of the base resistance has not been fully engaged. As mentioned previously, a higher wall-to-raft volume ratio is associated with higher aspect ratios, indicating an increase in shaft resistance. Consequently, BCR values tend to exhibit a steeper increase at lower load levels for higher aspect ratios.

The MBR and MSR values act as indicators for assessing the extent to which a BSDF exhibits monolithic behavior. As the MBR and MSR values approach one, it signifies that the BSDF behaves increasingly like a monolithic compound body. In the case of MBR, it represents the ratio between the bearing capacity of a BSDF and the bearing capacity of a typical embedded foundation. As for MSR, it signifies the ratio between the settlement of a BSDF and the settlement of an embedded foundation at the bearing pressure equivalent to the capacity of a representative BSDF.

The geometry of BSDFs has a notable influence on their monolithic behavior. Concerning depth, initially, MBR values deviate from unity as depth increases. However, once the depth reaches a certain point, MBR values begin to show a slight increase. As depth initially increases, MSR values undergo a significant rise and diverge from unity, but they level off once a certain depth is reached. With an increase in width, MBR values for equivalent depths tend to diverge from one initially. However, once a specific threshold is reached, this pattern changes, and MBR values undergo a notable increase, moving closer to unity. Conversely, MSR values demonstrate a consistent initial increase and approach unity as width expands. In general, BSDFs characterized by lower aspect ratios tend to demonstrate MBR and MSR values that are closer to one, indicating behavior more akin to a monolithic-compound body. Additionally, it has been observed that beyond a certain point, MBR and MSR values show a slight increase and then stabilize at a specific value.

BSDFs with rough interfaces tend to exhibit MBR and MSR values that approach one, indicating behavior more resembling a monolithic-compound body, in contrast to BSDFs with smoother interfaces. However, as R_{int} values increase, the impact of BSDF depth on both MBR and MSR values diminishes, resulting in reduced variability in these values.

The relative density of the underlying sand affects the monolithic behavior of BSDFs, although this influence may not be as readily noticeable as the impact on load-settlement behavior. Denser sands tend to demonstrate a higher degree of monolithic behavior, especially when lower aspect ratios are taken into account.

In order to consider a range of loading levels, MBR values were determined by examining bearing pressures at settlements corresponding to 1.25%, 2.5%, and 5% of the foundation's width. Lower aspect ratios generally lead to MBR values approaching one when load levels are higher, whereas higher aspect ratios result in MBR values approaching one at lower load levels. In other words, monolithic behavior increases with load level at small aspect ratios and decreases with load level at large aspect ratios.

It can be inferred that BSDFs with higher aspect ratios and relative densities may exhibit a more pronounced arching effect compared to BSDFs with lower aspect ratios and relative densities. However, it is crucial to remember that monolithic behavior depends on

various factors, including arching, the wall-to-raft volume ratio, as well as the degree of soil compression, which is influenced by geometry and surface roughness.

For CPRFs, parameters such as pile configuration, the number of piles, pile length, the ratio of raft-to-soil stiffness, load level and configuration can be varied. Like CPRFs, BSDFs also come in various geometries, with differences in box dimensions, including length and width, as well as variations in the number of boxes and the thickness of walls and rafts. Additionally, load levels, load arrangements, and soil types can vary, significantly impacting the performance of BSDFs. As a result, there is an extensive number of BSDF configurations that can be compared to CPRFs. Hence, the concrete volume of deep foundations serves as a reference parameter for conducting this comparison. The variable $P\%$ is used to denote this “efficiency” representing the percentage change in load capacity between BSDFs and CPRFs with the same volumes. The load capacity has been determined with a foundation settlement of 50 mm, aligning with the serviceability limit state. If $P\%$ is positive, it implies that, when comparing various deep foundations, BSDF exhibits a greater load capacity than CPRF when subjected to a 50 mm settlement.

In the context of this study, the initial comparison involved deep foundations with identical volumes and raft widths, followed by a subsequent comparison of deep foundations with the same volume but varying raft widths. The comparisons conducted with a constant total volume pertain to deep foundations of the same raft width, designed to emulate structural foundations for buildings. Conversely, those with varying raft widths were intended to represent structural foundations for bridges.

Firstly, a comparative assessment was conducted between CPRF configurations featuring four, 25, and 49 piles and BSDFs with equivalent volumes. This comprehensive analysis encompassed a total of 180 distinct models, encompassing various CPRF aspect ratios ranging from one to four. The parametric study, which involved comparing BSDFs and CPRFs with equal volumes, unveiled that the efficiency of BSDFs is primarily governed by their geometry. Consequently, formulating a sweeping conclusion regarding the efficiency of BSDFs across all scenarios is impractical. Nevertheless, the fundamental findings from this parametric investigation can provide valuable insights when making

decisions about which deep foundation configurations to consider during the initial phases of a project.

Among all deep foundation configurations, when the raft width increases, compared deep foundations typically exhibit positive $P\%$ values, indicating higher efficiency. Moreover, with the growing aspect ratio of corresponding CPRFs, $P\%$ tends to exhibit a broader range of values. Across all deep foundation configurations, following an aspect ratio that varies but is approximately one, an increase in the aspect ratio of a BSDF corresponds to an increase in the $P\%$ value.

The specific surface values for CPRF and BSDF configurations stand at four and 3.33, respectively. CPRFs, due to their larger specific surface area, inherently possess greater shaft resistance. Nevertheless, it is noteworthy that certain BSDF configurations with higher aspect ratios display positive $P\%$ values. BSDFs, with their lower specific surface area compared to CPRFs, may have compensated for this difference through tip resistance. The tip resistance denotes the monolithic behavior created through the combined influence of the tip portions of BSDF walls and the soil core. Remarkably, even under relatively low load conditions associated with a 50 mm settlement, findings indicate that this monolithic behavior can reach levels ranging from 70% to 80%.

Secondly, the efficiency of BSDF configurations was assessed by comparing them to CPRFs with the same volumes, each consisting of nine piles. These compared deep foundations exhibit different raft volumes and geometries, with adjustments made to the volumes of walls or piles to maintain an equal total volume. Efficiency values demonstrate a substantial increase as the aspect ratios of BSDFs rise, starting at -21.94% for an aspect ratio of 0.81 and peaking at 73% for an aspect ratio of 4.68. The positive $P\%$ values could be linked to the increasing BSDF aspect ratios, which result in a larger surface area, as well as the contribution of the soil core due to monolithic behavior.

In the scope of this study, rigid foundations were chosen, and total settlement values were taken into account. Additionally, it is evident that rigid raft foundations are less prone to differential settlements. In the case of heavily loaded areas on BSDFs, soil improvement

could be achieved through methods such as jet grouting, or piles could be strategically positioned beneath the raft to effectively manage and control differential settlements.

As further research, it is recommended to assess BSDFs with different geometries such as rectangular or circular shapes to consider the shape effects on the load-settlement behavior. In this study, poorly graded sand (SP) with three different relative densities ranging from 35 to 65% has been simulated. However, it is recommended to work with the cohesive and mixed soils to simulate the real in-situ behavior of BSDFs. Studies that account for both vertical and horizontal loads, as well as seismic loads, should be conducted to compare BSDFs and CPRFs. Consequently, along with concrete volume, the reinforcement ratio can also be considered as a comparison parameter.

Based on the efficiency concept outlined in this study, it can be inferred that BSDFs may serve as a viable alternative to CPRFs in various scenarios. Furthermore, their capacity to resist scouring effects for bridges and their potential utility as retaining structures for supporting the basement excavation pit represent additional appealing characteristics of BSDFs.

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