Numerical analysis of limited width dense gravel backfills for plane strain conditions

Mo'oud Nasr
Coffey Geotechnics, Markham, Ontario, Canada
Kyle M. Rollins
Department of Civil and Environmental Engineering – Brigham Young University, Provo, Utah, U.S.A

ABSTRACT
Two series of full-scale lateral pile cap tests are presented, which show an increased lateral passive resistance of the pile cap when a dense gravel zone with limited width is compacted near the cap. A two dimensional finite element program was used to simulate the plane strain passive behavior of the full-scale tests. Parametric studies were also performed to evaluate the influence of selected geometric and geotechnical parameters on the passive resistance of limited width gravel backfills. Numerical simulations were able to capture the passive response of homogeneous and limited width backfills reasonably well, in terms of horizontal and vertical movements and failure mechanisms. Numerical results also indicate that significant increases in passive resistance can also be expected for long abutment walls where end effects are less pronounced and the geometry is closer to a plane strain (2D) condition. Based on results obtained from the parametric studies, a design method is developed for predicting the ultimate passive resistance of limited width gravel backfills, for plane strain (2D) geometries.

RÉSUMÉ
Deux séries d'essais de tête de pieux à pleine échelle avec des charges latérales sont présentés, qui montrent une augmentation de la résistance passive latérale de la tête de pieu quand une zone d'une largeur limitée de gravier est compactée près de la tête de pieu. Un programme à deux dimensions d'éléments finis a été utilisé pour simuler le comportement passive de déformation plane des essais en grandeur réelle. Des études paramétriques ont également été réalisées pour évaluer l'influence de certains paramètres géométriques et géotechniques sur la résistance passive des remblais à largeur limitée de gravier. Des simulations numériques ont été en mesure de capter la réponse passive de la largeur homogène et limiter les remblais raisonnablement bien, en termes de mouvements horizontaux et verticaux et les mécanismes de défaillance. Les résultats numériques indiquent que augmentation significative de la résistance passive peut également être prévu pour les murs de butée long, où les effets sont moins prononcés et la géométrie est plus proche d'une condition de déformation plane (2D). Sur la base des résultats obtenus à partir des études paramétriques, une méthode de conception est développé pour prédire la résistance ultime passive des remblais à largeur limitée de gravier, pour les déformations planes (2D) géométries.

1 INTRODUCTION

One common approach for increasing the bearing capacity of spread foundations is to excavate and replace the weak soils with compacted granular fill. However, this approach has not been used to increase the passive resistance on an abutment or pile cap for lateral loadings. To investigate this approach, (Rollins et al. 2010) and (Gerber et al. 2010) conducted full-scale lateral pile cap tests in which a narrow, high density gravel zone was compacted adjacent to the cap with loose sand beyond the gravel zone. Homogeneous backfills consisting of loose sand and dense gravel were also tested, to quantify the effectiveness of limited width gravel backfills. These full-scale tests indicated that large increases (150%-250%) in lateral resistance, relative to homogeneous loose sand backfills, could be produced with relatively narrow dense gravel zones (54% to 163% of the pile cap height). This passive resistance was also about 50% to 70% of that expected if the gravel backfill extended far enough to encompass the entire failure surface. This increased lateral resistance is similar to the increased vertical bearing pressure obtained by placing compacted granular fill below a spread footing. Narrow gravel zones could be especially beneficial for increasing lateral earth pressure in cases where the full replacement of the backfill with select material would not be economically viable.

Although these tests confirm the practicality of the method, direct application of the test results is limited by several factors. First, the pile cap tests were performed for a limited number of pile cap and backfill geometries, but no standard methodology was provided to design for other geometries. Second, the field tests were performed on pile caps where 3D end effects were significant. It is unclear if the same increases would be obtained for long abutment walls, where end effects are less pronounced and the geometry is closer to a 2D or plane strain condition.

To address these limitations, plane strain finite element analyses were performed using the commercial computer software, PLAXIS 2D-Version 8 (Brinkgreve et al. 2005). The objective of the numerical analyses was to simulate the development of passive earth pressures observed during the field tests, in two dimensions, for the various backfill conditions that were tested. This would provide an evaluation of the effectiveness of limited width gravel backfills for long abutment walls, where end effects are less pronounced and the geometry is closer to a plane strain condition. To validate the analysis procedure, numerical results were first calibrated against analytical results obtained from PYCAP (Duncan and Mokwa 2001) and ABUTMENT (Shamsabadi et al. 2007) for homogeneous backfills with the log-spiral approach. The analytical models were additionally validated by comparison with measured ultimate passive resistances obtained by Rollins et al. (2010a) and Gerber et al. (2010).

Using the calibrated FEM model, a series of parametric studies was then performed to assess the impact of various soil and pile cap geometry parameters on the passive resistance of dense limited width backfills. Based on the parametric studies a simple design approach was developed that can be used as an aid in the design of limited width backfills for 2D (plane strain) geometries.

2 FULL SCALE TESTING

Two series of full-scale lateral pile cap tests that involved dense gravel backfills of limited width were reported by (Rollins et al. 2010a) and by (Gerber et al. 2010) at test sites located near the intersection of Interstate 15 and South Temple and at the Salt Lake City International Airport in Salt Lake City, Utah, respectively. Profile views of the test configuration at the South Temple and SLC Airport sites are illustrated in Figures 1 and 2. Each series of tests consisted of laterally loading a full-scale pile cap using hydraulic actuators and recording the passive resistance mobilized by a variety of homogeneous and limited width backfills. The main objective of the tests involving limited width gravel zones was to determine whether the narrow gravel zone would cause any increase in passive resistance.

2.1 South Temple Testing

The South Temple reinforced concrete pile cap was 3.05 m long, 5.18 m wide, and 1.12 m deep and was supported by a pile group consisting of 12 closed-ended steel pipe piles, with an outside diameter and wall thickness of 324 and 9.5 mm, respectively. The piles were driven to a depth of approximately 12.2 m below the ground surface with center to center spacings of 1.42 m and 1.06 m in the transverse and longitudinal directions, respectively. The concrete used in the cap had a compressive strength of 34.5 MPa. The steel reinforcement in the cap mainly consisted of a reinforcement mat with transverse and longitudinal reinforcing bars placed in both the top and the bottom of the cap.

The backfill conditions tested at the South Temple site included: (1) homogeneous loose sand; (2) limited width dense gravel backfill consisting of a 0.91-m wide zone of dense gravel between the pile cap and loose sand and; (3) limited width dense gravel backfill consisting of a 1.83-m wide zone of dense gravel between the pile cap and loose sand. The 5.18 m wide by 1.12 m deep side of the pile cap was backfilled from 0.3 m below the base of the pile cap to a height of approximately 1.12 m. The backfill extended approximately 4.9 m behind the pile cap and 1.8 m laterally beyond the edges of the cap on each side. The final dimensions of the backfill zone after placement were approximately 8.8 m wide and 4.9 m long.

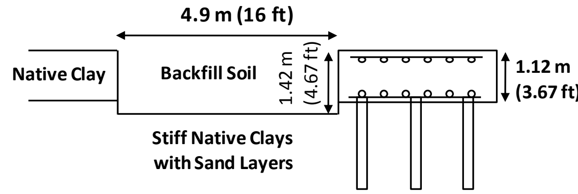


Figure 1. Profile view of the South Temple [5.18 m (17 ft) wide cap] test configuration.

2.2 SLC Airport Testing

The reinforced concrete pile cap at the SLC Airport was 1.68 m deep, 4.57 m long, 3.35 m wide and was supported by a pile group, consisting of 6 closed-ended steel pipe piles, with an outside diameter and wall thickness of 324 and 9.5 mm, respectively. The piles were driven to a depth of approximately 13 m below the ground surface, with a center to center spacing of 3.66 m in the direction of loading. The concrete used in the cap had a compressive strength of 41 MPa. The cap reinforcement mainly consisted of longitudinal and transverse reinforcing bars, placed in both the top and the bottom of the cap.

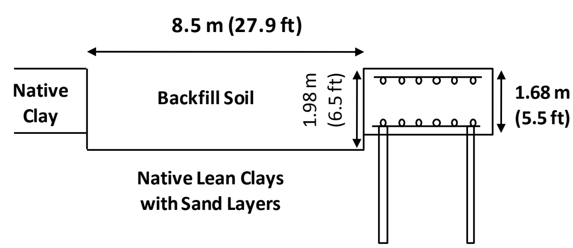


Figure 2. Profile view of the SLC Airport [3.35 m (11 ft) wide cap] test configuration.

The backfill conditions tested at the SLC Airport site include: (1) homogeneous loose sand; (2) homogeneous dense fine gravel; (3) limited width dense gravel backfill consisting of a 0.91-m wide zone of dense gravel between the pile cap and loose sand and; (4) limited width dense gravel backfill consisting of a 1.83-m wide zone of dense gravel between the pile cap and loose sand. The 3.35-m wide by 1.68-m high side of the pile cap was backfilled from 0.3 m below the base of the pile cap to a height of approximately 1.68-m. The final dimensions of the backfill zone after placement were approximately 7.0 m wide and 8.5 m long.

2.3 Backfill Soil Properties

Two different soil types were used as backfill material around the front of the pile cap: silty sand and fine gravel. According to the Unified Soil Classification (USC) System the silty sand classified as SM. The American Association of State Highway and Transportation Officials (AASHTO) classification of the material is A-4. The maximum particle size of the fill was 12.5 mm with approximately 90% passing the No. 40 sieve and 45% non-plastic fines content. The coefficient of uniformity, C_u, and curvature, C_c, were 14.8 and 2.8, respectively. The fine gravel used as the compacted fill was a typical roadbase material, which classified as silty, clayey gravel with sand (GC-GM) according to the USC System. The AASHTO Classification of the material is A-1-b. The gravel fill had a maximum particle size of 19 mm. C_u and C_c were 454 and 1.2, respectively. Index properties associated with the silty sand and fine gravel materials are summarized in Table 1.

Table 1: Summary of backfill soil index properties.

Backfill Type	Gravel (%)	Sand (%)	Fines (%)	C_u	C_c
Silty Sand	2.4	52.9	44.7	15	2.8
Fine Gravel	49.7	30.5	19.9	454	1.2

Several in-situ and laboratory direct shear tests provided estimates of the loose silty sand strength parameters. Strength parameters associated with the dense fine gravel were estimated based on direct shear tests performed on a comparable material at a different site. Nuclear density tests were performed during compaction on each layer of compacted silty sand and fine gravel fill to determine the average dry unit weight, $\gamma_{d\ (avg)}$.

Table 2: Summary of backfill soil engineering parameters.

Backfill Type	$\gamma_{d\ (avg)}$ kN/m^3	w_{avg} (%)	D_r (%)	ϕ (°)	c kPa	δ/ϕ
Loose Silty Sand	15.7	11.1	40	27.7	6.8	0.75
Dense Fine Gravel	20.8	6.1	85	42.0	19.6	0.75

The relative density (D_r) was estimated based on the average dry unit weight using correlations developed by Lee and Singh, (1971). The interface friction angle, δ, was determined by performing soil-concrete direct shear tests, as well as recommendations given by Potyondy, (1961). A summary of the engineering characteristics of the loose silty sand and dense fine gravel materials is provided in Table 2.

2.4 Measured Static Load-Displacement Response

Figures 3 and 4 provide plots of the backfill passive load-displacement curves associated with the South Temple and SLC Airport lateral pile cap tests, respectively. In each figure, the total measured passive force has been normalized by the actual pile cap width to obtain the force per width of pile cap. For the South Temple curves, the 0.91-m wide dense gravel zone increased the total static passive resistance by 75% to 150% relative to the homogeneous loose sand test, at any given pile cap deflection. In the case of the 1.83-m dense gravel zone, this increase is 150% to 225% relative to the homogeneous loose sand backfill.

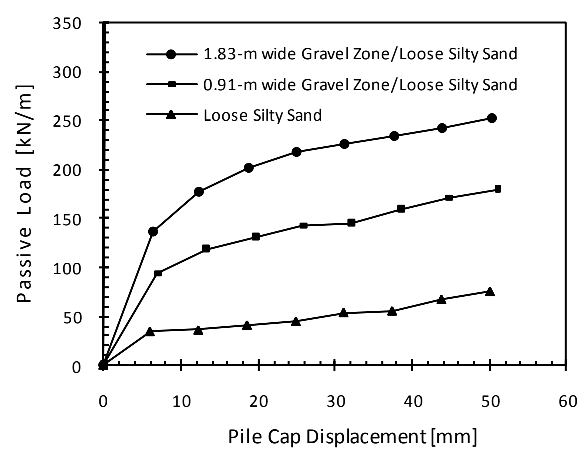


Figure 3. Comparison of measured load-displacement curves, normalized by the pile cap width of 5.18 m, for South Temple backfill conditions consisting of: (1) homogeneous loose sand; (2) 0.91-m wide gravel zone and loose sand; and (3) 1.83-m wide gravel zone and loose sand.

In addition, the limited width dense gravel backfills tested mobilized a significant portion of the resistance that would have been developed if homogeneous dense gravel backfills were used instead. The 0.91-m and 1.83-m wide dense gravel zone and loose sand backfills mobilized 54% and 78% of the passive resistance associated with homogeneous dense gravel backfills, respectively.

In the case of the SLC Airport tests, placement of either a 0.91-m or 1.83-m wide zone of dense gravel between the pile cap and loose sand backfill increased the total static passive resistance of the limited width backfills by approximately 300%, relative to the homogeneous loose sand backfill at a displacement level of 45 mm. This passive resistance with the limited width backfills is about 60% of the resistance that developed for the homogeneous dense gravel backfill.

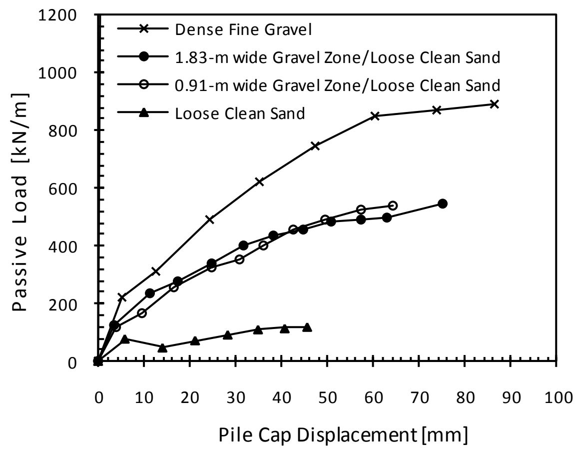


Figure 4. Comparison of measured load-displacement curves, normalized by the pile cap width of 3.35-m, for SLC Airport backfill conditions consisting of: (1) homogeneous loose sand, (2) homogeneous dense gravel, (3) 0.91-m wide gravel zone and loose sand, and (4) 1.83-m wide gravel zone and loose sand.

3 NUMERICAL ANALYSIS

3.1 2D Plaxis Model

The finite element modeling program PLAXIS 2D-Version 8 was used to simulate the passive behavior of a selected range of homogeneous and limited width backfill conditions tested at both South Temple and SLC Airport sites under static loading conditions. An individual beam element with linear-elastic behavior was used to model the reinforced concrete pile cap. To simplify the model further, the piles were omitted from the finite element model and their effect was considered by prescribing a zero vertical displacement boundary on the pile cap. This simplification in representing the pile foundation system was justified based on the fact that the measured vertical movement of the concrete cap supported by pile foundations was minimal in the field.

In addition, a maximum prescribed horizontal displacement corresponding to a deflection-to-wall height ratio of 4% was applied to pile caps with homogeneous and limited width backfill conditions involving dense gravel. In the case of the homogeneous loose sand backfill, a maximum prescribed horizontal displacement corresponding to a deflection-to-wall height ratio of 6% was applied.

3.2 Backfill Soil Input Parameters

The Hardening Soil constitutive model (Shanz et al. 1999, Brinkgreve et al. 2005) was employed in approximating backfill soil behavior and interface joint elements were used to approximate the interaction between the soil mass and the adjacent pile cap with elastic-plastic behavior.

The initial estimates of the input parameters used to model the loose sand, dense gravel, and the interaction between the pile cap and the adjacent backfill, were derived from laboratory-based measurements and correlations. These parameters were further adjusted iteratively by matching load-displacement curves computed numerically with curves measured experimentally and computed analytically with PYCAP and ABUTMENT. Both PYCAP and ABUTMENT calculate the ultimate passive force using the log spiral method and compute the load-displacement curve using a hyperbolic curve. Table 3 provides a summary of the calibrated soil parameters used in the numerical analysis.

Table 3: Summary of Hardening Soil model input parameters.

Parameter	Loose Sand	Dense Gravel
Friction angle, ϕ [°]	27.7	42.0
Cohesion, c_{ref} [kPa]	0.5	1.9
Dilation angle, ψ [°]	0	12
Soil unit weight, γ_m [kN/m^3]	17.3	22.1
Secant stiffness modulus, E_{50}^{ref} [MPa]	15.8	81.4
Reference stress, P_{ref} [kPa]	100	100
Poisson's ratio, υ_{ur}	0.2	0.2
Interface strength reduction factor, R_{inter}	0.7	0.7

During this iterative process, soil parameters which appeared to have the greatest influence on the predicted ultimate passive resistance included the soil friction angle ϕ, cohesion c, dilation angle ψ, and the interface strength reduction factor R_{inter} defined by equation 1.

$$R_{inter} = \tan\delta / \tan\phi \quad [1]$$

where δ is the wall/interface friction angle. Other parameters such as the soil stiffness value E_{50}^{ref} controlled the steepness or slope of the hyperbolic load-displacement curve. In this study the wall friction was taken as 75% of the soil friction angle based on testing conducted by Potyondy (1961).

Values of soil friction angle, φ, and cohesion intercept, c, were primarily selected based on in-situ and laboratory direct shear test measurements for the loose silty sand and dense fine gravel materials. However, to provide a more general application of the numerical results obtained in this study, a cohesion value close to zero was used in PLAXIS, enough to produce sufficient numerical stability.

The secant stiffness parameter corresponding to a stress level of 50% of the ultimate stress, at a reference stress equal to 100 stress units, E_{50}^{ref} is the main input stiffness parameter used in PLAXIS. E_{50}^{ref} of 15.8 and 81.4 MPa were selected iteratively at a reference stress equal to atmospheric pressure 100 kPa for the loose sand and dense gravel materials, respectively. This selection was based on providing agreement (within 10%) between numerical and analytical results. The selected E_{50}^{ref} values also compare well with the range of initial stiffness modulus values E_i, recommended by Duncan and Mokwa, (2001) for shallow foundations on granular soils (E_{50}^{ref} values are approximately 70-80% of E_i values provided by Duncan and Mokwa, 2001, at a confining stress equal to atmospheric pressure).

The dilation angle, ψ, was computed using equation 2 as recommended by Brinkgreve et al. (2005).

$$\psi \approx \begin{cases} \phi - 30^\circ, & for\ \phi \geq 30^\circ \\ 0^\circ, & for\ \phi \leq 30^\circ \end{cases} \qquad [2]$$

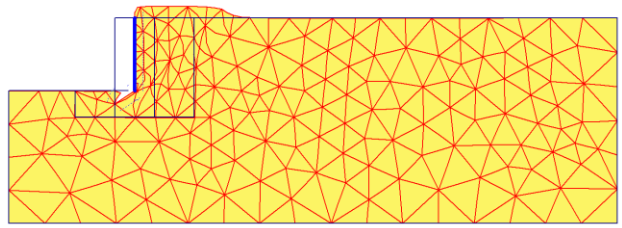
3.3 Numerical Analysis Results for Geometries Comparable to Field Tests

Following the calibration of numerical models, associated with homogeneous backfills, against load-displacement curves generated by PYCAP and ABUTMENT, the calibrated model was then employed to analyze the passive resistance of limited width dense gravel backfills, tested experimentally. Numerical simulation results presented in this section include total displacements, shear strains, and load-displacement curves.

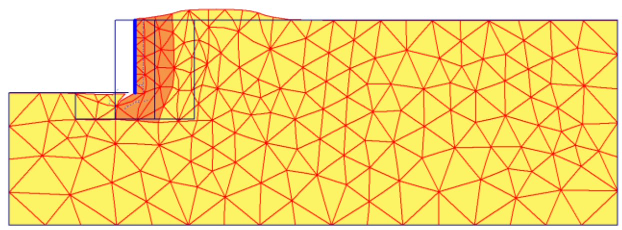
3.3.1 Total Displacements

Figure 5 shows deformed finite element mesh profiles of the simulated backfill conditions for the 1.68-m deep pile cap. For the sake of visualization, these profiles have been magnified five times from their true scale. The lowest soil layer boundary is 3.0 m from the base of the pile cap, and the silty sand layer extends horizontally to a distance of 11.6 m in front of the pile cap. Boundary conditions were defined at each geometry point by prescribing a known force or displacement. The standard fixity boundary condition available in PLAXIS was applied to the nodes at the three sides of the soil mass. This option creates fixities in the horizontal and vertical directions at the horizontal boundary of the geometry, and rollers on the two vertical boundaries. An ultimate prescribed displacement boundary condition was applied to the pile cap with no vertical displacement.

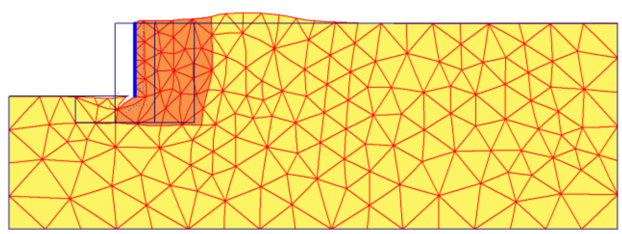
It should be observed that a significant amount of movement is predicted by the numerical models near the top of the pile cap for the homogenous backfills, where the soil has heaved upward owing to the lateral deflection of the pile cap. This observation is consistent with field measurements for the homogeneous dense gravel backfill tested at the SLC Airport site, where upward movement begins adjacent to the cap. Further comparison of deformed mesh profiles associated with homogeneous backfills, indicates that the zone of heaving is longer for the dense gravel backfill than for the loose sand backfill owing to the longer shear surface resulting from the higher friction angle. This observation is consistent with measurements from the SLC Airport tests.



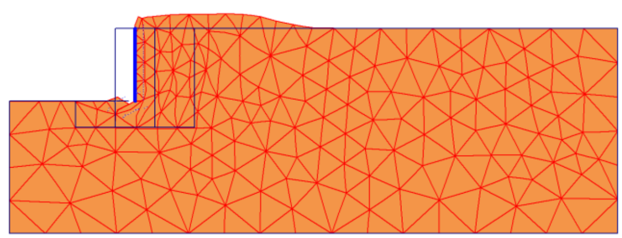
(a) Homogenous loose sand backfill



(b) 0.91-m gravel and loose sand backfill



(c) 1.83-m gravel and loose sand backfill



(d) Homogenous gravel backfill

Figure 5. Deformed mesh profiles of 1.68-m deep pile cap with backfills consisting of: (a) homogeneous loose silty sand; (b) 0.91-m wide dense gravel zone and loose silty sand; (c) 1.83-m wide dense gravel zone and loose silty sand; and (d) homogeneous dense gravel.

For the limited width gravel zones, the numerical model predicts that the dense gravel zone will deflect into the loose sand layer with relatively little heaving in the gravel, but that increased heaving would be expected just beyond the loose sand boundary. Vertical movements measured for limited width backfills tested experimentally show similar behaviors. The shift in elevation between the gravel zone and loose silty sand boundary may possibly be an effect of the pile cap stresses being transmitted through the gravel zone into the loose silty sand portion of the limited width backfill. Greater lengths of heaving are also predicted in the loose sand for the 0.91-m wide gravel zone than for the 1.68-m wide gravel zone. This is presumably due to the reduced pressures at the 1.68-m interface compared to the 1.12-m interface.

In addition to displacements at the ground surface, major movements are concentrated at the base of the pile cap, in the deformed mesh profiles, where the shear zone displaces the soil. This observation emphasizes the importance of ensuring that the compacted dense gravel fill extends beneath the pile cap to intercept the shear zone, particularly for gravel zones of limited width. Sensitiviy analyses indicate that the gravel should extend 0.6 m below the base of the cap.

3.3.2 Incremental Shear Strains

As the pile cap translates horizontally into the soil mass under the application of the prescribed displacements, the soil fails in shear along a critical failure surface behind the pile cap. This shear surface can be defined by a band of high shear strains and large incremental displacements from the computer output. To provide insight into the geometry of the potential shear surfaces developed in the analyzed backfills, incremental shear strain contours are illustrated in Figure 6, showing the shear patterns associated with the failure of the backfills.

For homogeneous loose silty sand backfills, the observed failure surface, resulting from possible punching shear behavior of the 1.12-m and 1.68-m deep pile caps, initiates from the base of the caps and extends outward in an approximately linear manner, until it intersects the ground surface. This is similar to a typical planar failure surface assumed in the Rankine theory of passive earth pressure. Shear strain contours of backfills involving dense gravel, show a more curvilinear failure surface, similar to a typical log spiral failure surface observed in dense gravels. The curved log spiral portion of the failure surface, initiates from the base of the pile cap, dipping approximately 0.6 m beneath the base of the pile cap, before it extends linearly to intersect the ground surface.

Another interesting aspect of shear strain patterns shown in Figure 6 is that for the 1.12-m deep pile cap, the failure surface appears to remain well within the gravel zone, as the gravel zone width increases, thereby providing greater passive resistance. In contrast, for the 1.68-m deep pile cap, even though the main portion of the shear zone passes through the gravel zone, shear strain concentrations appear to accumulate and extend around the gravel zone. In this case, a smaller percentage of the failure surface would be contained in the gravel zone, relative to the 1.12-m deep pile cap, as the gravel zone width increases, resulting in relatively lower gains in passive resistance. This observation may be a possible explanation for the differences in resistance observed between the limited width backfills tested at the South Temple and SLC Airport sites noted previously. In addition, as mentioned in the previous section, this phenomenon emphasizes the importance of ensuring that the compacted dense gravel fill extends sufficiently beneath the pile cap to intercept the shear zone developed in limited width dense gravel backfills.

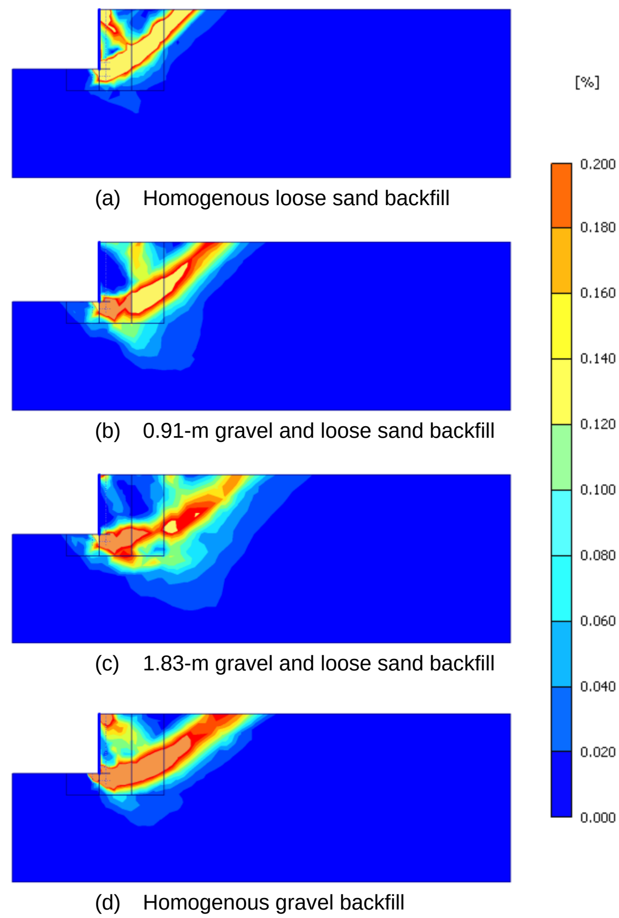


Figure 6. Incremental shear strain profiles of 1.68-m deep pile cap with backfills consisting of: (a) homogeneous loose silty sand; (b) 0.91-m wide dense gravel zone and loose silty sand; (c) 1.83-m wide dense gravel zone and loose silty sand; and (d) homogeneous dense gravel.

3.3.3 Load-Displacement Curves

Figure 7 illustrates the effectiveness of limited width backfills in increasing the plane strain passive resistance for the South Temple pile cap geometry. Relative to the homogeneous loose sand backfill, placement of the 0.91-m and 1.83-m wide dense gravel zones between the pile cap and loose sand increased the passive resistance of the backfill 84% and 152%, respectively relative to the loose sand backfill. In addition, the 0.91-m and 1.83-m wide dense gravel zones mobilized 43% and 59%, respectively, of the passive resistance provided by the homogeneous dense gravel backfill. Note that the increases in 2D resistances associated with the South Temple pile cap geometry are lower than the measured increases associated with the 3D case which was presented previously in Figure 3. This result is expected, as 3D end effects were anticipated to provide a significant component of the experimentally observed increase in passive resistance mobilized by the backfill. These 3D end effects increase the effective width of the pile cap thereby increasing the observed passive resistance whereas this effect is not accounted for in the 2D analysis.

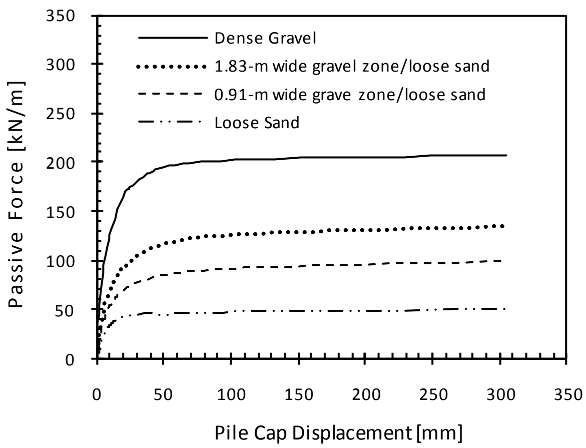


Figure 7. Comparison of load-displacement curves computed with PLAXIS for 1.12-m deep pile cap with backfills consisting of: (1) homogeneous dense gravel (2) homogeneous loose sand (3) 0.91-m wide gravel zone and loose sand; and (4) 1.83-m wide gravel zone and loose sand.

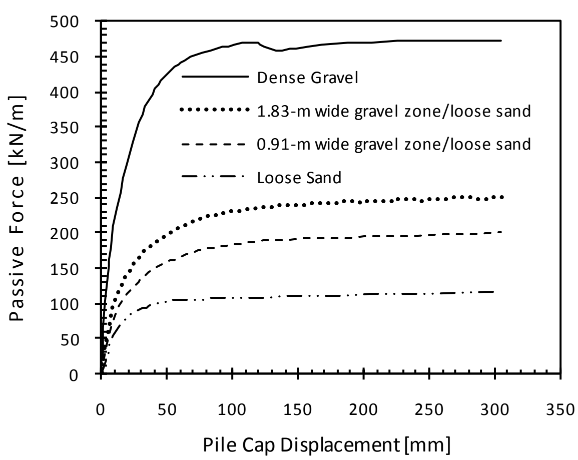


Figure 8. Comparison of load-displacement curves computed by PLAXIS for 1.68-m deep pile cap with backfills consisting of: (1) homogeneous dense gravel; (2) homogeneous loose sand; (3) 0.91-m wide gravel zone and loose sand; and (4) 1.83-m wide gravel zone and loose sand.

Figure 8 illustrates the effectiveness of limited width dense gravel backfills in increasing the ultimate plane strain passive resistance for the SLC Airport pile cap with a higher wall height. Relative to the homogeneous loose sand backfill, placement of the 0.91-m and 1.83-m wide dense gravel zones between the pile cap and loose sand increased the passive resistance of the backfill 60% and 100%, respectively. In addition, the 0.91-m and 1.83-m wide dense gravel zones mobilized 38% and 48% of the passive resistance associated with the homogeneous dense gravel backfill, respectively. These results indicate that the dense gravel zones were less effective in increasing passive resistance as the cap height increased. Similar to the South Temple pile cap geometry, the increases in 2D resistances associated with the SLC Airport tests are lower than for the 3D case.

3.4 Parametric Studies

To better understand the factors affecting the increased passive resistance, a series of parametric studies were executed on the limited width backfill numerical models. It was found that in a limited width dense gravel backfill, the ultimate passive force is significantly influenced by the wall height H, gravel friction angle ϕ_g, and the friction angle associated with the loose sand portion of the limited width backfill, ϕ_s.

Typical pile cap heights analyzed in the parametric studies were within the range of 0.9 to 2.4 m. Limited width backfill conditions used in the assessment include the following: (1) homogeneous loose sand; (2) limited width dense gravel backfill consisting of a 0.91-m wide zone of dense gravel between the pile cap and loose sand and; (3) limited width dense gravel backfill consisting of a 1.83-m wide zone of dense gravel between the pile cap and loose sand. Typical gravel friction angles analyzed in the parametric studies were 35°, 39°, and 42°. For the looser sand portion of the backfills, friction angles of 27.7°, 32°, and 36° were used in the analysis.

Results obtained from these studies were used to develop a simple design equation that can be used as an aid in designing limited width backfills for plane strain geometries. This equation is expressed as follows:

$$PFR = 3.418 - 0.139\,\phi_s - 0.033\,\phi_g + 0.484(B_F/H) - 0.043(B_F/H)^2 + 0.003\,\phi_s^2 - 0.007\,\phi_s(B_F/H) \qquad [3]$$

The passive force ratio (PFR) is defined as the ratio of the mobilized passive resistance in a limited width dense gravel backfill, P_{LW}, over the mobilized passive resistance of a homogeneous dense gravel backfill, $P_{FW\text{-}Gravel}$. PFR is a function of the sand friction angle, ϕ_s (in degrees), gravel friction angle, ϕ_g (in degrees), and the gravel zone width normalized by the pile cap height, B_F/H. Relative to the passive force ratios predicted by the numerical simulations, equation 3 has an absolute percentage under-prediction error of 8% in an extreme case and a maximum over-prediction error of 4%. However, predicted passive force ratio values within the 25th and 75th percentile fall in an error range of -2 to +1%.

It is important to emphasize that the results presented in this section have been developed based on plane strain numerical simulations of limited width gravel backfill conditions, tested experimentally. Under this assumption, the contribution of 3D edge effects on the passive resistance of the analyzed backfills is ignored, and the simulations carried out do not simulate the actual 3D passive response of the full-scale tests. As such, Equation 3 serves as a guide for the plane strain approximation of the mobilized passive resistance in limited width backfills, and is only applicable to situations in which applying plane strain conditions is a reasonable assumption. An example of this condition would be a relatively long abutment wall where the edge effects have negligible impact on the passive resistance mobilized in the adjacent backfill. In addition results presented in this section are valid under the assumption that the depth of gravel zone treatment extends 2 ft (0.61 m) below the base of the pile cap, and that the pile cap would be capable of tolerating movements equal to 4% for limited width backfills.

3.5 Conclusions

Based on the experimental test results and numerical analysis performed on the South Temple and SLC Airport pile caps, the following conclusions can be made:

1. The plane strain numerical simulations were able to capture the passive response of homogeneous and limited width backfills reasonably well, in terms of horizontal and vertical movements and failure mechanisms.

2. Predicted heaving profiles and shear shading plots show that major horizontal movements and strains are concentrated at the base of the pile cap, where the shear zone displaces the soil. This observation emphasizes the importance of ensuring that the compacted dense gravel fill extends beneath the pile cap to intercept the shear zone, particularly for gravel zones of limited width.

3. For homogeneous loose silty sand backfills, the failure surface resembles a typical planar failure surface assumed in the Rankine theory of passive earth pressure. Homogeneous dense gravel backfills and the limited width backfills, show a more curvilinear failure mechanism, which is similar to the log spiral failure surface assumed in the log spiral theory of passive earth pressure.

4. For the 1.12-m deep pile cap (South Temple), with increasing width of the gravel zone, the failure zone appears to remain well within the gravel zone, providing greater passive resistance with increasing width of the gravel zone. In contrast, for the 1.68-m deep pile cap (SLC Airport), the failure surface appears to extend below the bottom of the gravel zone, with increasing width of the zone. In this case, a smaller percentage of the failure surface would be contained in the gravel zone, relative to the shorter pile cap, reducing the effectiveness of the compacted fill in increasing the passive resistance of the backfill.

5. Limited width dense gravel backfills increased the plane strain ultimate passive resistance of the backfills, considerably, compared to the homogeneous loose silty sand backfill. Furthermore, the plane strain ultimate resistance mobilized in the limited width dense gravel backfills constituted a significant portion of the passive resistance that would have been provided, if a homogeneous dense gravel backfill had been used. This result indicates the effectiveness of using limited width dense gravel backfills, despite the relatively narrow width of the dense gravel zones placed between the pile cap and loose silty sand in comparison to the length of the log spiral failure surface.

6. Parametric studies show that the wall height H, gravel friction angle ϕ_g, the friction angle associated with the loose sand portion of the limited width backfill, ϕ_s, and the strength reduction parameter, R_{inter}, have a relatively significant effect on the passive resistance mobilized by limited width dense gravel backfills. Based on these results, an appropriate selection of these parameters is important in providing an accurate assessment of the expected passive resistance.

7. Equation 3 provides a simple estimation of the plane strain passive resistance of limited width dense gravel backfills. This equation was developed based on plane strain numerical simulations of full-scale limited width backfill conditions, tested experimentally, and thereby account for important geotechnical design parameters.

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