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ABSTRACT

Faced with difficult soil conditions for the support of two 6-story office towers in Dublin, CA and of a 6-story parking garage in Sacramento, CA, engineers recommended the use of Geopier Rammed Aggregate Piers to reinforce the soil for the support of high bearing capacity spread footings instead of deep foundations. Foundation selection for both sites was influenced by long-term settlement performance, schedule and cost savings, and seismic uplift resistance. Rammed aggregate piers were installed to strengthen upper weak and compressible soil layers resulting in a substantially stiffer soil layer on which shallow, high bearing capacity spread footings were constructed. Several rammed aggregate piers were also installed with steel anchors to resist seismically induced overturning forces. Design parameter values were confirmed by full scale aggregate pier modulus tests and uplift tests and a 24 hour load test at the Dublin site. Total settlements were estimated to be less than 1½ inches for the Sacramento site and less than 1 inch for the Dublin site. Measured settlements are less than 1 inches total for both projects with differential settlements less than ½ inches, confirming the design approaches and soil properties used for design. Site selection, rammed aggregate pier design methodology, modulus and uplift load test results, and measured settlement performance are presented for two projects in California.

INTRODUCTION

The support of buildings using shallow foundations is generally the first consideration of geotechnical engineers when preparing foundation recommendations because of construction costs, reliable performance, and ease of construction. However, geotechnical engineers are often faced with poor soil conditions that can increase construction costs. In the past decade, Geopier rammed aggregate piers (RAP) have gained wide acceptance for strengthening and reducing the compressibility of soft clays, undocumented fills, and loose sands.   The implementation of Geopier RAPs in California's seismic Zones 3 and 4 requires the use of uplift elements to resist seismically induced overturning forces.

The two case histories discussed illustrate the performance offered by RAP support of shallow high bearing capacity footings. These case histories also illustrate that the use of the RAP foundation system resulted in cost savings within the same performance standards as other conventional foundation systems. RAP design methodology, construction, modulus and uplift tests, and measured settlements are presented.

SACRAMENTO, CA - PROJECT DESCRIPTION

The structure built by McCarthy Building Companies, Inc. is a 6-story parking garage with Class A offices on the first floor,

see Fig.1. The Block 224 garage, located in Seismic Zone 3, was built for the State of California as part of the Capitol Area East End Project in 2000. The garage houses 753 parking spaces and offices on half of the first floor.

Kleinfelder Inc. of Sacramento performed a site investigation and identified compressible alluvial soil to depths of 30 and 35 feet. Kleinfelder recommended 75 foot long driven concrete piles to control excessive building settlements because conventional spread footings could not meet a settlement requirement of less than 1.5 inches. McCarthy wanted to reduce costs and schedule to win the design-build bid for the

Fig.1. 1301 P Street Parking Garage, Sacramento, CA

project and considered RAP soil reinforcement supporting high bearing capacity footings as a design-build foundation alternative to the driven pile design. The RAP foundation system was estimated to save at least 60 days on the schedule and $100,000 in construction costs. McCarthy contracted Consolidated Engineering Labs of Sacramento (CEL), to evaluate the use of the rammed aggregate piers for the project. McCarthy was awarded the project and CEL required full-scale load testing prior to completing the foundation design.
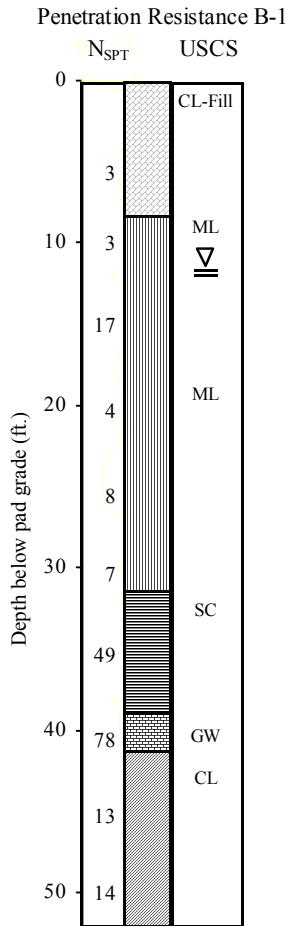

Fig.2. Sacramento Site - Soil Profile

The site is underlain by undocumented fill to depths of 7 feet, then alluvial, soft to medium silty clay and sandy silt to 30 feet, then alluvial, dense sand and gravel to about 42 feet, then medium to stiff clay to about 65 feet and then dense gravels to the maximum depth explored of 80 feet. Figure 2 shows the soil profile and standard penetration resistance to a depth of 50 feet near the RAP modulus load test location. The upper undocumented fill was placed during the late 1800's and early 1900's to raise the city above flood waters. Groundwater was observed between 8 and 12 feet below the ground surface during RAP installations. Bounded on three sides by city streets and an alley on the fourth, the site is located in a busy section of downtown Sacramento. Because RAP construction

produces much less noise and vibrations than pile driving, local businesses were less impacted during RAP installation. Construction was set to begin in February 1999, historically one of the wettest months of the year in the area.

Building dead plus live loads for the cast-in-place concrete structure ranged from 138 to 835 kips on gravity columns and 1600 to 3600 kips at seismic resisting shearwalls. Net uplift forces at shearwall ends equaled a maximum of 740 kips.

DUBLIN, CA - PROJECT DESCRIPTION

The structures built by DPR Construction Inc. of Redwood City, CA consist of two identical 6-story Class A offices with a pedestrian bridge in between, Fig. 3. Located in Seismic Zone 4 and in an old army depot, the site had undocumented fills and old building foundations beneath the ground surface.

Fig.3. 6-Story Class A Offices in Dublin, CA

The project was planned to get started early in December 2000. DPR and the developer were looking for ways to reduce costs and build the project through the winter. They considered RAP soil reinforcement with high bearing capacity spread footings for the 6-story steel moment frame buildings.

Kleinfelder Inc. of Pleasanton, CA had performed a site investigation which revealed compressible clay soil to depths of 50 feet. Kleinfelder recommended three foundation options: 1) 7 feet of overexcavation and recompaction with conventional spread footings, 2) 65 foot long pre-cast concrete driven piles and grade beams, and 3) RAPs with high bearing capacity spread footings. Figure 4 shows a comparison of the three options. The grading option was deemed to be risky
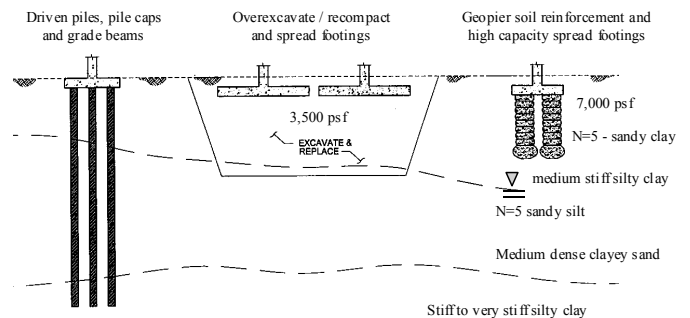

Fig.4. Foundation systems considered for the Dublin site

because of wet weather conditions. The cost for piles and grade beams was high compared to the rammed aggregate piers and footings. RAP and high bearing capacity spread footings were selected by the developer and DPR because of performance, cost savings, and schedule advantages.

The Dublin site was underlain by undocumented clayey fill with old shallow concrete foundations to depths of 7 feet, then by 12 to 16 feet of medium stiff silty clay, then by soft and stiff silty clay to the maximum depth explored of 50 feet. Figure 5 presents the soil profile, Standard Penetration Test resistance, and Cone Penetration Tip resistance near the RAP load test site. Groundwater was observed at depths of 22 feet during installation of rammed aggregate piers.

Building dead plus live loads for the steel moment frame structure ranged from 260 to 970 kips. Being in Seismic Zone 4, maximum service level seismic forces were as high as 1,010 kips. Net uplift forces equaled 599 kips at the buildings core moment frames.
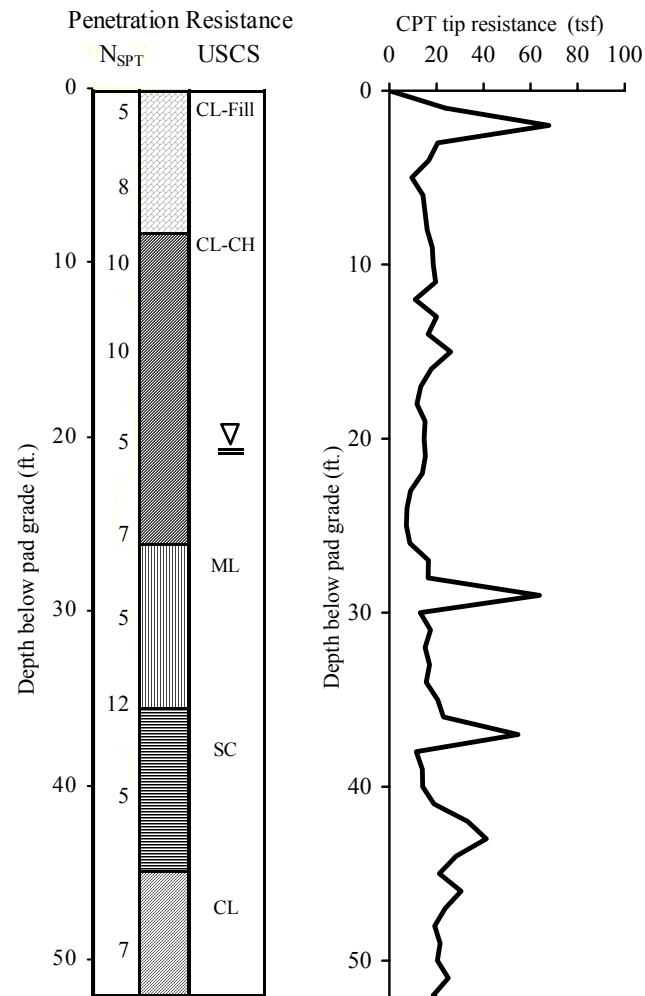
RAMMED AGGREGATE PIER CONSTRUCTION

The sequence of RAP construction is shown in Fig. 6. RAPs are installed by drilling 24-, 30-, 33-, and 36-inch diameter shafts and ramming thin 12-inch lifts of well-graded, crushed, aggregate in the shafts to form very stiff, high-density aggregate piers. The ramming equipment consists of 18 ton to 27 ton hydraulic excavators equipped with 2,000 to 4,000 pound hydraulic break hammers and specially modified beveled tampers. The hydraulic hammer delivers between 1 to 2 million ft.-lbs. of ramming energy per minute to the beveled tamper at 300 to 500 blows per minute. Figure 7 shows the typical installation equipment. The ramming action increases the lateral stress in the surrounding soil and increases the stiffness of the stabilized composite soil mass.
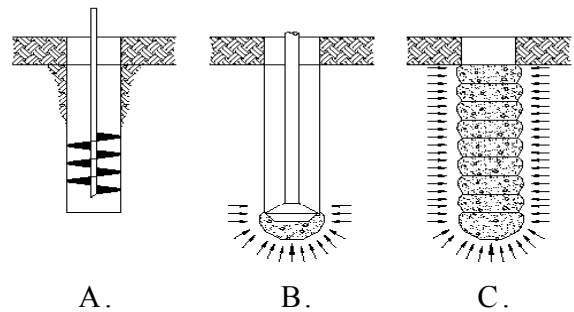

Fig.6. Rammed Aggregate Pier Construction Process
    A. Drill 30-inch and 33-inch RAP shafts
    B. Ram 2-inch crushed rock into bottom bulb
    C. Ram 12-inch lifts of 3/4 or 1½ inch aggregate base rock to 6" above the design elevation

The beveled tamper densifies, forces, and embeds the crushed aggregate laterally into the sidewalls of the shaft. The result of RAP installation is a significant strengthening and stiffening of subsurface soil. In addition, high lateral stresses and shear strengths are able to support high bearing capacity spread footings with minimal settlement. Rammed aggregate piers exhibit friction angles varying from 48 to 52 degrees and dry densities from 140 to 148 pcf (Fox and Cowell 1998).

Depending on pier depth, soil conditions, and construction site conditions, typical RAP installation rates vary from about 35 to 60 elements per day. Slower rates are typical for installations that require temporary casing. A discussion of quality control testing for rammed aggregate pier installations can be found in the ICBO ES Report ER-5916 (2002).

RAP installations were performed during the winter months of 1999 and 2001 at the two sites. Two 22 ton hydraulic excavators equipped with 3,500 pound hydraulic break hammers and specially modified beveled tampers were used to ram the aggregate lifts at both sites.

At the Sacramento site, drill depths below pad grade were 20 feet for 36-inch RAP uplift elements and ranged between 11 and 20 feet for 30-inch RAP bearing elements. Crews

Fig.5. Dublin Site – Soil Profile

Fig.7. Typical RAP Installation Equipment

Fig.8. Upper Zone and Lower Zone for Spread Footing

installed 428 RAPs at the Sacramento site during inclement weather in February and March 1999. McCarthy was able to begin excavation and construction of spread footings after about 50% of the rammed aggregate piers were installed.

At the Dublin site, drill depths below pad grade were 22 feet for 33-inch RAP uplift elements and ranged between 11 feet and 25 feet for 30-inch RAP bearing elements. Seven hundred and twenty-eight RAPs were installed during winter weather conditions at the Dublin site in January and February of 2001.

RAMMED AGGREGATE PIER DESIGN CALCULATIONS

Rammed aggregate pier construction increases vertical and horizontal stresses in the matrix soil and increases the stiffness of soil and fill which significantly reduces foundation settlements (Lawton and Fox 1994, Pitt et al. 2003). In high seismic and high wind zones, the resistance to lateral and uplift forces is required. Brief discussions of RAP design methodologies for settlement control, uplift resistance, and lateral resistance are presented and comparisons of calculated to measured settlements are discussed.

Total Settlement Control

Design calculations for estimating total settlement of shallow foundations supported on RAPs are well described in the literature (Lawton and Fox 1994, Lawton et al. 1994, Fox and Cowell 1998, Minks et al. 2001, Hall et al. 2002). The design procedure computes foundation settlements by considering settlement in the upper, RAP reinforced zone and settlements in the lower, unreinforced zone of soil below the RAP bottom bulb or prestress zone. Figure 8 shows the upper zone (UZ) and lower zone (LZ) in section. The total settlement is computed as the sum of the upper zone and lower zone settlement values.
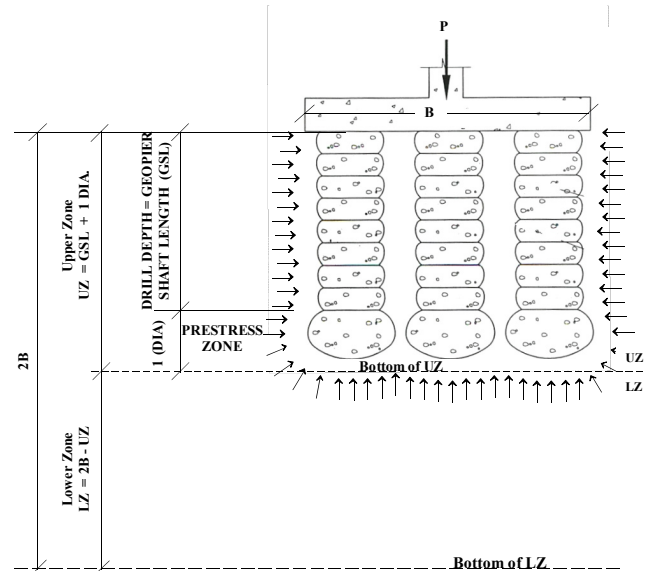
Upper zone settlement calculations

As compressive loads are applied to RAP supported footings, the stiff piers attract a greater portion of footing-bottom stress than the softer matrix soil. The distribution of stress depends on the ratio of stiffness of RAPs to matrix soil and on the ratio of area of the RAPs to the gross footing bottom area. Upper zone calculations are based on a spring analogy (Lawton and Fox 1994 and Lawton et al. 1994) and are described in the following equations:

1. Footings are assumed to be perfectly rigid relative to the foundation materials. Thus, the stresses applied to the composite foundation materials depend on their relative stiffnesses and area coverage. From static equilibrium, the total load on the footing (P), expressed as the product of applied composite stress (q) and footing area (A), is resisted by a total upward resisting force in the rammed aggregate piers (Q_g) and soil (Q_s) materials:

$$P = qA = Q_g + Q_s = q_g A_g + q_s A_s \qquad (1)$$

where q_g is the stress on top of the RAP, A_g is the area of the RAPs below the footing, q_s is the vertical stress on the matrix soil, and A_s is the area of the matrix soil below the footing.

2. The settlement of the RAP will equal the settlement of the matrix soil due to the rigid footing condition. The upper zone settlement (s_{uz}) of the foundation(s) can be written in terms of RAP top stress (q_g) and stiffness modulus (k_g) or in terms of the matrix soil stress (q_s) and soil stiffness modulus (k_s):

$$s_{uz} = q_g/k_g = q_s/k_s \qquad (2)$$

3. Equation 2 can be rewritten to express the matrix soil stress in terms of the RAP top stress and the ratio of the pier and

matrix soil stiffness modulus values by expressing the stiffness ratio (R_s) as $R_s = k_g/k_s$:

$$q_s = q_g(k_s/k_g) = q_g/(k_g/k_s) = q_g/R_s \qquad (3)$$

4. Combining Equations 1 and 3 and by expressing the area ratio (R_a) as $R_a = A_g/A$ and rewriting qg in terms of q:

$$q_g = [qR_s / (R_aR_s+1-R_a)] \qquad (4)$$

Upper-zone settlements are computed using Equations 2 and 4 which depend on the applied footing stress, the relative stiffness of the RAP and matrix soil, the area ratio of the RAPs, and the RAP stiffness modulus. RAP stiffness modulus values are measured in the field with full scale modulus load tests. Table 1 presents typical values of k_g and allowable RAP and soil composite bearing capacity (q_c) based on the standard penetration resistance and undrained shear strength of unimproved soil or fill.

Table 1. RAP stiffness modulus - $k_g{}^1$ and composite q_c

N_{spt}	S_u	k_g/q_c for Silty Sand and Sand	k_g/q_c for Silt and Clay	k_g/q_c for Peat
N	(psf)	(pci) / (ksf)	(pci) / (ksf)	(pci) / (ksf)
3	500	165 / 5.0	125 / 4.5	75 / 3.5
6	1,250	225 / 6.0	175 / 5.0	110 / 4.0
9	1,750	260 / 7.0	210 / 6.0	125 / 5.0
12	2,300	285 / 8.0	250 / 7.0	-
16	3,000	310 / 8.5	260 / 7.0	-
25	4,500	325 / 9.0	275 / 7.5	-
>25	5,000	360 / 10.0	300 / 8.0	-

1. For 30-inch diameter RAPs supporting spread footings with a minimum Ra = 30% (Cowell and Fox 1998).

Lower Zone Settlement Estimates

Estimates of lower zone settlements below the bottom of the rammed aggregate pier bulb are computed using conventional geotechnical settlement analysis procedures well described in the literature and texts (Terzaghi and Peck 1967 and Bowles 1988) combined with soil elastic modulus values interpreted from the results of in-situ testing data or from the results of laboratory oedometer consolidation tests. Lower zone calculations are also based on a distribution of stress radiating from the perimeter of the RAP zone of improvement, which can extend 0.5 to 1 diameter from the drilled edge, depending on the soil type. A Westergaard or Boussinesq analysis (Bowles 1988) is used for estimating lower zone footing stress influence factors (I_s).

Based on the soil conditions for the Sacramento site (Kleinfelder 1998), a RAP stiffness modulus of 350 pci and a high allowable dead plus live bearing capacity of 7,200 psf were used for design. Table 2 shows the RAP upper zone settlement design calculations for a 710 kip gravity column.

Table 2: Design Calculations for Footing M-11 at Sacramento

Calculation / Property	Eq. / Symbol	Value
Column D+L load (kips)	P	710
Footing width (ft)	B	10
Ave. bearing pressure (ksf)	$q = P/B^2$	7.1
No. of RAP	N_p	7
RAP diameter (ft)	d	2.5
Area replacement ratio (R_a)	$R_a=N_p(d/2)^2/B^2$	0.34
RAP stiffness modulus, (pci)	k_g	350
Soil stiffness modulus, (pci)	k_m	10.4
Stiffness ratio	$R_s=k_g/k_m$	34
RAP top-stress, q_g (ksf)	$qR_s/(R_sR_a-R_a+1)$	19.6
UZ settlement (in)	$s_{uz}=q_g/k_g$	0.39
Rap shaft length (ft)	H_s	12
UZ thickness (ft)	$H_{uz} = H_s+d$	14.5
Zone of footing influence (ft)	$H_t = 2 B$	20
LZ thickness (ft)	$H_{lz} = H_t - H_{uz}$	5.5
Mid-depth of LZ (ft)	$z = H_{uz} + H_{lz}/2$	17.25
Normalized LZ depth (ft)	z/B	1.73
Westergaard influence factor	I_s	0.1
LZ settlement (in)	Consol eqtn	0.45
Total footing settlement (in)	$S_{est} = s_{uz} + s_{lz}$	0.84
Max. actual settlement (in)	S_{actual}	0.54

Based on the soil conditions for the Dublin site (Kleinfelder 2000), a RAP stiffness modulus of 255 pci and a high allowable dead plus live bearing capacity of 6,500 psf were used for design. Table 3 shows the RAP settlement design calculations for a 499 kip gravity column.

Table 3: Design Calculations for Footing M5A at Dublin

| Calculation / Property	Eq. / Symbol	Value
Column D+L load (kips)	P	499
Footing width (ft)	B	9
Ave. bearing pressure (ksf)	$q = P/B^2$	6.2
No. of RAP	N_p	5
RAP diameter (ft)	d	2.5
Area replacement ratio (R_a)	$R_a=N_p(d/2)^2/B^2$	0.30
RAP stiffness modulus, (pci)	k_g	255
Soil stiffness modulus, (pci)	k_m	10.0
Stiffness ratio	$R_s=k_g/k_m$	25.5
RAP top-stress, q_g (ksf)	$qR_s/(R_sR_a-R_a+1)$	18.6
UZ settlement (in)	$s_{uz}=q_g/k_g$	0.51
Rap shaft length (ft)	H_s	14
UZ thickness (ft)	$H_{uz} = H_s+d$	16.5
Zone of footing influence (ft)	$H_t = 2 B$	18
LZ thickness (ft)	$H_{lz} = H_t - H_{uz}$	1.5
Mid-depth of LZ (ft)	$z = H_{uz} + H_{lz}/2$	17.25
Normalized LZ depth (ft)	z/B	1.92
Westergaard influence factor	I_s	0.08
LZ settlement (in)	Consol eqtn	0.36
Total footing settlement (in)	$S_{est} = s_{uz} + s_{lz}$	0.87
Max. actual settlement (in)	S_{actual}	0.75
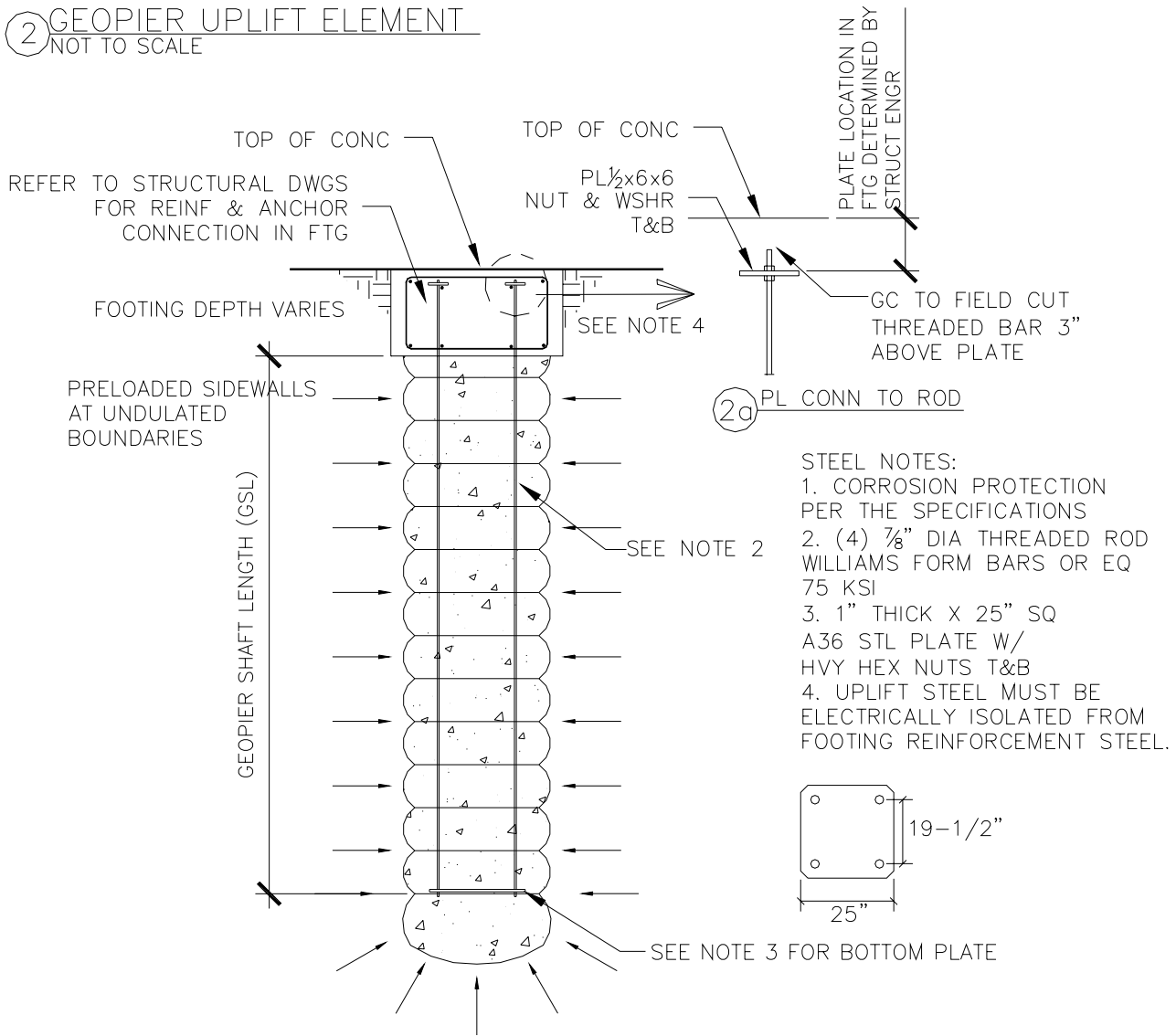
Fig.9. Typical Detail of a 4-bar RAP Uplift Element

RAP Uplift Elements

Rammed aggregate pier uplift elements were installed to resist seismic overturning and uplift forces. An uplift anchor consists of two or four 75 ksi threaded steel bars bolted to a 1-inch thick A36 steel plate. Design lives greater than 100 years are achieved with oversized bars, special poly coatings, galvanization, and electrical isolation from the footing reinforcement. Uplift resistance is developed by perimeter shearing resistance along the element and is enhanced by the high lateral stresses at the edges of the shaft (Wissmann et. al. 2001 and Caskey 2001). Figure 9 shows a typical detail for a 4-bar RAP uplift element. The design procedure computes the unit resistance to vertical movement (f_s) as the product of the effective horizontal earth pressure ($\sigma_{h'}$) and the tangent of the unimproved soil friction angle ($\emptyset'_s$) or as its undrained shear strength (s_u):

for cohesionless soil - $f_s = k_p \sigma_{v'} \tan(\emptyset'_s)$ (6)
for cohesive soil - $f_s = s_u$ (7)

where $\sigma_{v'}$ is the effective vertical stress and k_p is the Rankine passive pressure coefficient (Lawton et al. 1994). The ultimate uplift capacity (T_{ult}) is computed by integrating the unit uplift resistance (f_s) over the perimeter area (A_s) of the RAP plus the weight of the pier (W_{pier}):

$$T_{ult} = f_s A_s + W_{pier}$$ (8)

Typical allowable capacities of 50 to 80 kips are developed. The allowable capacity is typically increased by a factor of one-third or more for seismic loads. The ultimate uplift capacity of RAPs were computed using Equations 6, 7 and 8 and confirmed by performing uplift load tests at the two sites.

RAP Elements and Lateral Resistance

Rammed aggregate piers attract a greater amount of stress because they are stiffer than the matrix soil under applied loads (Pitt et al 2003). The combined high friction exhibited by RAPs and the high normal forces on the piers increases the ultimate sliding coefficient (f_{ult}) for the entire footing bottom. The design procedure used to estimate the ultimate sliding friction of a RAP supported footing considers the area ratio, the stiffness ratio, and the RAP friction angle ($\varnothing'_g$), Eq. 8.

$$f_{ult} = \frac{[R_s R_a \tan(\varnothing'_g)] + [(1-R_a)\tan(\varnothing'_g)]}{(R_a R_s + 1-R_a)} \quad (8)$$

A detailed discussion of the statics behind Eq. 8 can be found in Wissmann et al. 2001. A safety factor between 1.5 and 2.0 is typically applied for design to obtain the allowable sliding coefficient (f_{all}). The allowable load resistance is typically increased by a factor of one-third or more for seismic loads.

Table 4. Typical values of f_{all} for RAP/soil composites

Soil Classification	Typical $\varnothing'_s$	f_{all}[1]
Sand and Gravel	28° - 45°	0.52 – 0.60
Silt and Clay	20° - 30°	0.51 – 0.52

1. Values computed for R_s = 15, R_a = 33%, and FS = 2

RESULTS OF CONSTRUCTION

Modulus Test Configurations

Figure 10 shows a modulus test section and a photo of the test set up. The test set up consists of a compression element, two uplift elements, and a reaction frame. The compression element is loaded to 150% of the maximum top-of-pier stress calculated from Eq. 5. The load is applied against the reaction frame and resisted by the uplift elements. A telltale is installed at the bottom of the modulus test pier, just above the bottom bulb, to facilitate the measurement of bottom-of-pier deflections. During testing, the deflections at the top and bottom of the RAP are measured.

Modulus (Compression) Test Results

Figures 11 and 12 present the results of the modulus tests performed at the Sacramento and Dublin site respectively. The purpose of the modulus test is to verify the RAP stiffness modulus (kg) used for design calculations in Eq.(s) 2 and 4 at 117% of the design top-of-pier stress, qg. Table 3 presents the tested RAP deflections, tested top-of-pier stress, and tested RAP stiffness modulus for both sites.

Although the purpose of the modulus test is to verify the RAP stiffness modulus used for design calculations, the tests may also be used to add insight into how the RAP behaves in the
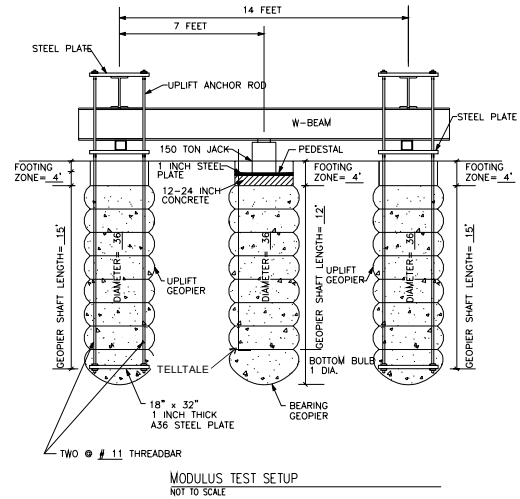

Fig.10. Section and Photo of RAP Modulus Test Set-up

matrix soil. This is done by observing the deflections of telltales installed into the bottoms of the RAPs. As shown on Fig.12 at the Dublin site, the bottom of the test pier is shown to move only slightly while deformations at the top of the pier increase at a growing rate. This behavior is interpreted to indicate that the RAP is bulging outward.

A 24 hour test was performed at the Dublin site, where the load of 117% times q_g (105 kips) was applied for 24 hours. The total deflection after 24 hours was 0.03 inches.

Table 3. Results of Modulus Tests for both sites

Sacramento site: 36-inch dia. x 12 foot GSL Design top-of-pier stress (q_g) = 18,224 psf		
117% x (q_g)	Deflection	Tested Modulus
21,322 psf	0.29 inches	510 psi/inch

Dublin site: 30-inch dia. x 18 foot GSL Design top-of-pier stress (q_g) = 18,649 psf		
117% x (q_g)	Deflection	Tested Modulus
21,819 psf	0.23 inches	670 psi/inch

Fig.11. Modulus test results for Sacramento site

Fig.12. Modulus Test Results graph for Dublin Site

Uplift (Tension) Test Results

The results of the uplift tests for both sites are presented in Fig. 13. At the design uplift load of 83 kips for the Sacramento site, a deflection of 0.19 inches was measured. At the design uplift load of 75 kips for the Dublin site, a deflection of 0.37 inches was measured. Temporary elongation of the 75 ksi threaded bars accounts for about one-half of the measured deflection which is supported by the rebound and the final deflection shown in Fig. 13. Uplift tests are performed up to 200% of the allowable uplift capacity.

Fig.13. Uplift Load Test Results at both sites

MEASURED SETTLEMENT PERFORMANCE

Two sites with compressible undocumented fill and soft clay soil were reinforced with rammed aggregate piers to support high bearing capacity footings. The measured results of total settlements after the addition of live loads are presented.

Sacramento Site

McCarthy was able to begin excavation and construction of concrete spread footings after about 50% of the RAPs were installed. As the first floor columns of the concrete structure were being poured, initial baseline top-of-footing surveys were measured. Once the sixth floor pour was completed, brass settlement monitoring monuments were set at 12 locations which included gravity columns with dead plus live loads ranging from 138 kips to 835 kips and at two shearwalls with dead plus live loads of 1,200 and 1,800 kips at the each end. Measurements were obtained until settlements flattened after 3 years from initial baseline readings.

The results of foundation settlement surveys are plotted against time in Fig.14. The results indicate that the foundation settlements have ranged between 0.25 and 0.75 inches with both the maximum value and the average of the values less than the design estimates.
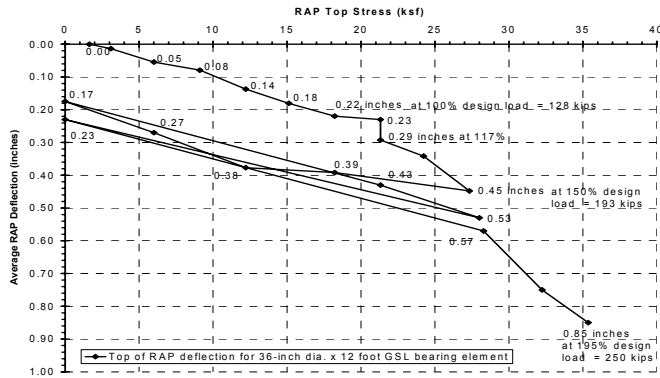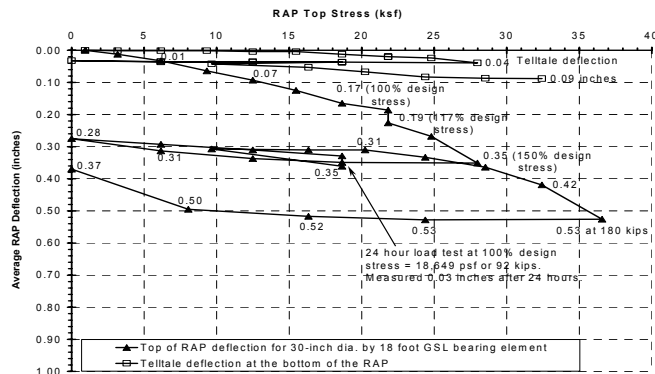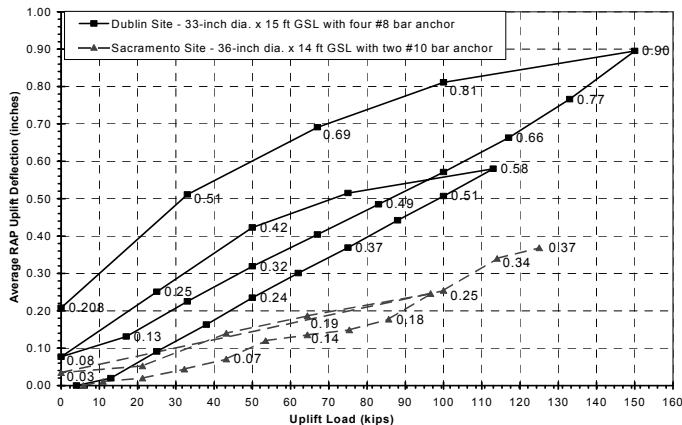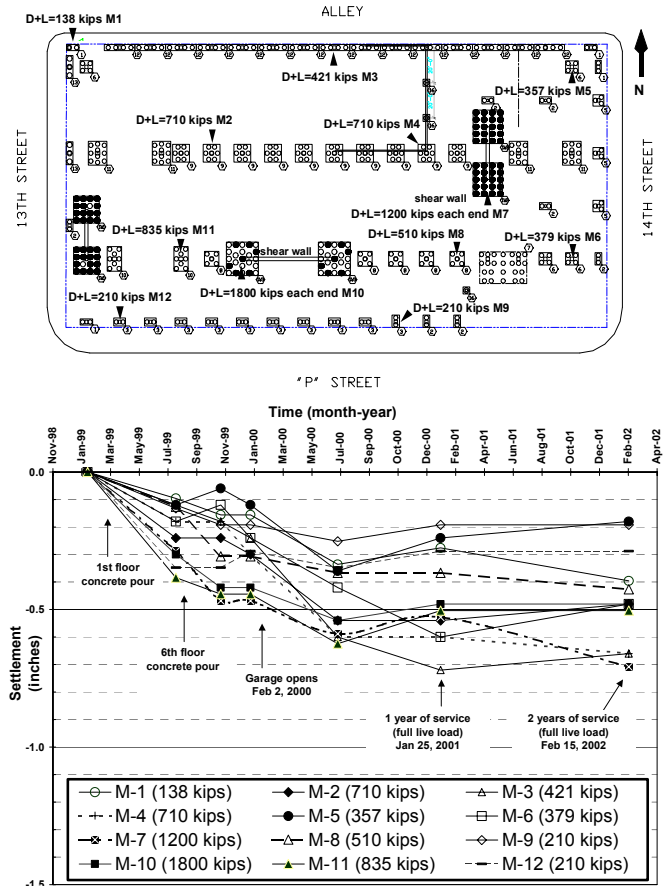

Fig.14. Surveyed locations and settlements Sacramento site

The footing settlement calculated in Table 4 was 0.84 inches and actual measured settlements were about 0.54 inches. If the tested modulus value of 510 pci is used then the calculated and measured settlements are in closer agreement.

Dublin Site

The Dublin project started at the end of December 2000 during winter rains. Load tests were performed during inclement weather for about a week. Settlement surveys started after erection of 3rd and 5th floor steel before concrete decks were poured. This accounts for about 20% to 25% of the dead loads. Several locations where monitored including gravity columns with dead plus live loads ranging from 300 kips to 600 kips and at moment frame mats with dead plus live loads of 1,500 and 2,300 kips. Settlements were measured up to the end of construction and once more after live load was applied.

The results of foundation settlement readings for the Dublin site are plotted against time in Fig.15 The footing settlement calculated in Table 5 was 0.87 inches and actual measured settlements ranged between 0.7 to 0.75 inches. The measured settlements are in good agreement with the estimated values.
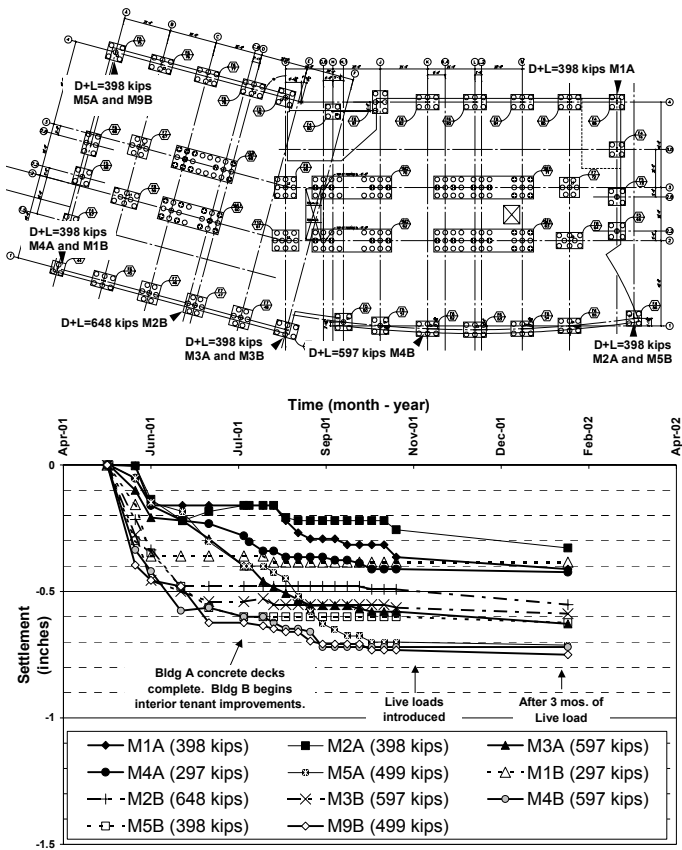

Fig. 15. Footing plan and settlements Dublin site

SUMMARY AND CONCLUSIONS

Two projects at soft soil sites in Sacramento and Dublin, California, utilized the Geopier rammed aggregate pier system to support shallow high bearing capacity spread footings for six-story building structures. The projects and the soil conditions have been described. The RAP elements were installed to average depths of 16 feet below footing bottoms to reinforce the undocumented fill and soft to medium-stiff clay and natural clay soils at both sites. The RAP foundation system replaced 65 and 75 foot concrete driven pile designs. Settlement surveys conducted during and after the construction of the buildings revealed that the RAP-supported footings settled ¼ to ¾ inches under the applied total loads. The measured settlement values are in good agreement with or less than the design estimates.

Explanations to why the measured settlements are lower than predicted values include: 1) the real benefit from increased lateral stress in the reinforced soil is not accounted for in RAP design procedures (Handy 2001), 2) the reduction in vertical stress to the lower zone due to the positive group interaction effects of the RAP system are not included in the design, 3) conservative estimates were made for the consolidation behavior of the lower zone soil, 4) predicted upper zone settlements are based on lower "more conservative" RAP stiffness modulus values when compared to actual tested modulus values, and 5) secondary compression in the lower zone may still occur over time.

These case histories illustrate that the use of RAP soil reinforcement to support high bearing capacity footings resulted in cost savings within the same performance standards as other conventional foundation systems.

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Symbols used in order of appearance:

P	dead plus live load downward force on a footing
q	applied bearing pressure
A	area of the footing bottom
Q_g	load resisted by rammed aggregate pier
Q_s	load resisted by soil
q_g	top stress on rammed aggregate pier
A_g	area of rammed aggregate piers below footing
q_s	bearing stress on soil
A_s	area of soil below footing
s_{uz}	upper zone settlement
k_g	stiffness modulus of rammed aggregate pier
k_s	stiffness modulus of unimproved soil
R_s	stiffness ratio
R_a	area ratio
N_{spt}	standard penetration test blow counts
s_u	undrained shear strength
H_s	length of drilled shaft below footing bottom
H_{uz}	thickness of upper zone soil
H_t	thickness of total zone of stress influence
H_{lz}	thickness of lower zone soil
Is	stress influence factor at mid-depth of lower zone
f_s	vertical rammed aggregate pier shaft resistance
$\sigma_{h'}$	effective horizontal earth pressure
k_p	Rankine horizontal earth pressure coefficient
$\sigma_{v'}$	vertical effective stress
ϕ'_s	effective soil friction angle
T_{ult}	ultimate uplift resistance
W_{pier}	weight of rammed aggregate pier
$\phi_{g'}$	rammed aggregate pier friction angle