Uplift Testing of Rammed Aggregate Pier® Systems

Tom Farrell[1] GE, MASCE, Brendan FitzPatrick[2] PE, William Kenney[3] GE MASCE.

[1]President, Farrell Design-Build Companies, Inc., 685 Placerville Drive, Placerville, CA 95667 USA, tom@farrellinc.com. [2]Director of Engineering, North America Geopier Foundation Company, Inc., 150 Fairview Road, Suite 335, Mooresville, NC 28117, USA, bfitzpatrick@geopier.com. [3]Project Manager, Farrell Design-Build Companies, Inc., 685 Placerville Drive, Placerville, CA 95667 USA, bill@farrellinc.com.

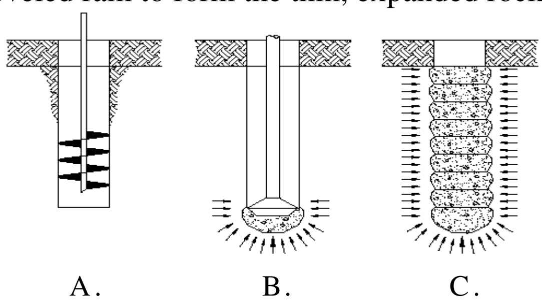
ABSTRACT: Recent changes and consolidation in building codes across the U.S. have put a larger emphasis on the need for cost-effective foundation support of both bearing and uplift foundation loads resulting from wind and seismic forces. The Rammed Aggregate Pier (RAP) system is a commonly-accepted soil reinforcement method that results in highly-densified aggregate columns for liquefaction mitigation, bearing capacity enhancement, and uniform settlement control for conventional shallow foundations. The RAP system provides a cost beneficial alternative to deep foundations such as pre-cast driven piles, auger-cast piles, or concrete piers and to mass excavation and replacement for shallow foundations. In high seismic and wind areas, uplift loads created by overturning forces on foundations are resisted with uplift RAPs that are equipped with a structurally designed steel anchor assembly. Typical uplift capacities in California range from 178 to 445 kN (40 to 100 kips) for 762 to 838 mm (30 to 33 inch), 3.6 to 7.0 m (12 to 23 feet) meter long RAP elements using ASD (allowable stress design) and incorporating a minimum factor of safety of 2. Full scale load tests on uplift RAPs show repeatable tangent stiffness after multiple overstress load cycles. This paper presents the first published results of psuedo-cyclic uplift load tests on uplift RAPs installed on projects in California.

INTRODUCTION

The nationwide adoption of the International Building Code in the US and recently in California presents challenges to engineers now faced with often higher seismic and wind forces updated by current events and research. Increased seismic forces in California are of particular concern in building design, construction, and performance. Seismic forces resulting in large short term bearing and uplift loads on a building foundation can be challenging and expensive to solve in California. This paper discusses the engineering, construction, performance, and load test results of Uplift Rammed Aggregate Pier systems (uplift RAPs). The limit states and mechanisms considered for friction capacity along the sides of the RAP shaft are discussed along with the structural design considerations of the steel anchors. Of particular importance, the results of multiple pseudo-cyclic uplift load tests are presented and discussed for differing soil conditions in California.

RAMMED AGGREGATE PIER CONSTRUCTION

Rammed aggregate pier systems are constructed by installing thin lifts of highly-densified aggregate using vertical ramming energy. Two different installation methods are typically used depending on the site specific soil conditions. The first method is a replacement RAP method (aka Geopier® RAP). This method involves removing soft and weak soil or fill by drilling shafts measuring 762 to 838 mm (30 to 33 inch) in diameter to depths ranging from 2 to 9 m (6 to 30 feet), see Figure 1. After drilling is complete, aggregate is added to the shaft and rammed in place. The ramming equipment consists of a 200 kN (45,000 pound) hydraulic excavator equipped with a 15.6 kN (3,500 pound) hydraulic break hammer and a specially designed 45º beveled ram to form the thin, expanded rock lifts (Majchrzak, 2004).



A. Drill 762 to 838 mm (30 and 33 inch) diameter shafts to design depth
B. Ram 51 mm (2 inch) crushed rock into the “bottom bulb”, then ram thin lifts
C. Ram 19 mm (¾ inch) crushed rock in 305 mm (12 inch) lifts to form a Geopier RAP.

Fig. 1 Geopier Rammed Aggregate Pier Construction Process

The second method is a displacement RAP method (aka Impact® RAP) which densifies and displaces soil by driving a hollow pipe mandrel into the ground. Aggregate is introduced to the bottom of the displaced hole through the pipe mandrel. The mandrel is raised and then driven back down to vertically densify thin lifts of aggregate. Aggregate and sand densification is achieved during installation using both vertical static crowd pressure and vertical impact ramming energy delivered by a high-frequency, vibratory, pile driving hammer. This RAP method is unlike vibro-replacement stone column methods where horizontal vibrations, that are produced by eccentric weights in a vibrating probe, are used to compact the stone. Displacement RAP elements are typically 508 to 610 mm (20 to 24 inch) in constructed diameter and extend to depths ranging from 3 to 15 m (10 to 50 feet). For greater densification and improvement, re-ramming of rock lifts often results in diameters over 762 mm (30 inches). The ramming equipment consists of a 445 to 624 kN (100 to 146 kip) piling rig equipped with a 1,350 kN (152 ton) vibratory hammer, pipe mandrel, and an expanded beveled ram. See Figure 2 for the Impact RAP method of construction.

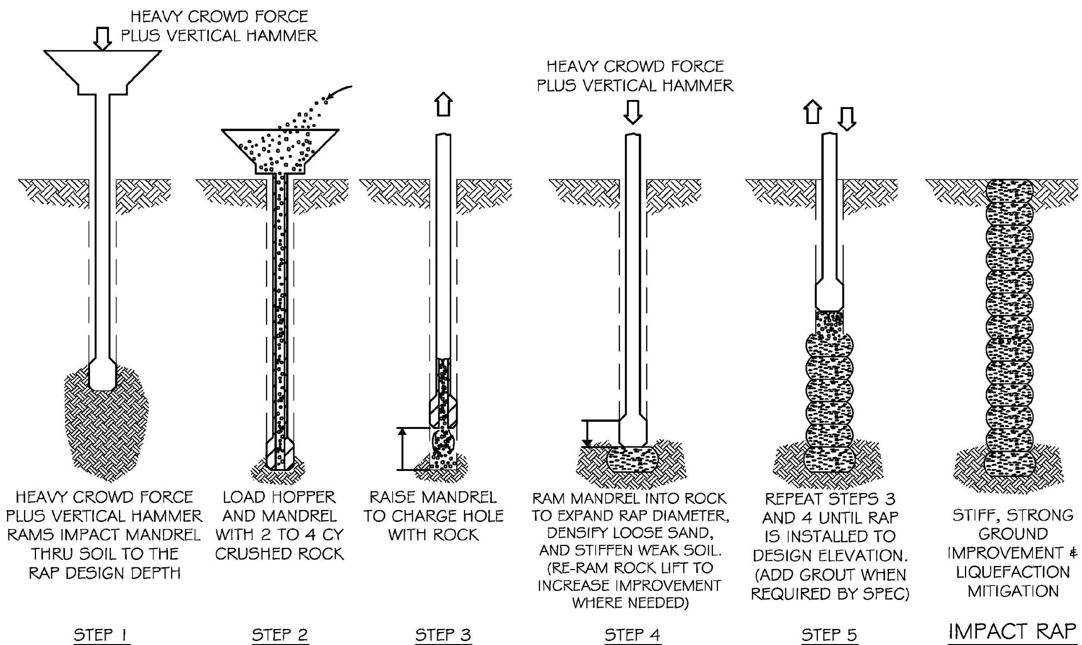


Fig. 2 Impact Rammed Aggregate Pier Construction Process

As a result of heavy crowd force and vertical ramming during installation, both RAP construction methods result in expansion of the aggregate at the edge of the pier (cavity expansion). These installation processes and resulting cavity expansion form undulated sides along the pier perimeter and significantly increase the lateral stress in the matrix soil. The combination of high lateral stress and undulated shape results in enhanced coupling of the RAP aggregate to the matrix soil providing an efficient mechanism for shear resistance along the sides of the RAP element (Handy 2001, White et al 2002). This development of high shear resistance is the basis for providing both high bearing and uplift resistance using RAPs for foundation support.

Both RAP construction methods deliver uplift resistance with the addition of a structurally designed steel anchor assembly. An uplift RAP is constructed by installing a vertical “dead-man” anchor at the “bottom bulb” elevation during construction of the RAP. In California, the structural steel anchor typically consists of two bard for Impact RAPs or four bars for Geopier RAPs. Williams Form Engineering #7 all-thread rebars with a minimum ultimate strength of 267 kN (60 kips) are used in most uplift RAP applications. Figure 3 shows an uplift anchor assembly and properties of the all thread rebar.

At the bottom of the anchor, the all thread bar are bolted to a 25 mm (1-inch) thick, hot dip galvanized, A36 steel plate. For permanent installations, the uplift assembly is corrosion protected. A robust corrosion protection system is used in California as follows: 1) oversized and hot dip galvanized steel bars, 2) asphaltic coating on the bars, 3) a heat-shrink polyurethane coating of 0.8 to 1.6 mm (30 to 60 mils) thick over the asphalt coating on the bars. The 25 mm plate nut assembly are also covered with a poly urethane coating. The tops of the all thread bar receive 102 mm (4 inch) square bearing plates, which provide the necessary anchorage in the concrete footing.

All Thread Structural Properties	
Minimum Yield Stress	Ultimate Tensile Stress
517 MPa (75 ksi)	698.5 MPa (100 ksi)

Fig. 3 RAP Uplift Assembly and All Thread Rebar Properties and Threads

UPLIFT RAP PERFORMANCE

Uplift RAP strength and stiffness performance is affected by several design parameters and the construction process. The most considered performance parameter by practicing engineers is typically "embedment depth," which the authors have observed by numerous load tests does not solely control the ultimate capacity and performance of the uplift RAP. The basic design and construction parameters to be considered for reliable uplift RAP strength and stiffness are:

1. Density of Rammed Aggregate directly above the bottom plate
2. Embedment depth "D" (distance between bottom of footing and bottom plate)
3. Shear resistance by skin friction in sand or undrained shear strength in clay
4. Stiffness of steel rods ($\Sigma AE/D$)
5. Group effects (multiple uplift RAPs)
6. Adjacent bearing RAPs (which increase confinement)
7. Number of load cycles
8. Uplift load

Expected seismic performance requires an equivalent spring stiffness from between 5.7 to 56.5 kN/m (50 to 500 kips/inch). Current uplift RAP design methods are based on well accepted limit states of embedment depth and shear resistance at the shaft edges. However, these limit states are based on strength and not deflection. Based on the performance of numerous load tests, the authors have observed that the parameter with the most influence on uplift RAP stiffness is the density and expansion of the rammed aggregate directly above the bottom plate. This parameter is also the most sensitive to the construction process and requires a specially trained and highly experienced operator on the ramming equipment.

Based on the author's observation and experience with numerous uplift RAP tests, the soil strength and rammed aggregate within the bottom 1/3 of the RAP often limits the RAP deflection response to load. Engineers can use judgment during design to increase the RAP tension capacity by specifying cement treated aggregate. When a stiffer response is needed or softer soils are observed within 1.5 m (5 feet) of the bottom plate, cement can be mixed with the aggregate and installed on and above the plate. The cement treated aggregate is placed from to 0.9 to 1.5 m (3 to 5 feet) above the plate, where vertical stresses induced in the pier from the applied uplift loads are highest – on the order of 1,915 kPa (40 ksf).

UPLIFT RAP ENGINEERING CALCULATIONS

The design for uplift RAPs must consider the limit states for the system. These limit states may be divided into 1) geotechnical design considerations which incorporate not only the behavior of the RAP element, but also relate the anticipated performance to the site-specific soil conditions as well as 2) structural design considerations focusing on the structural design of the high-strength rods and anchor plate. These limit states are described below.

Geotechnical Limit State Considerations

Uplift RAPs generate capacity by developing shear resistance along the perimeter of the shaft. The shear resistance is enhanced by the high lateral stress imparted by cavity expansion at the edges of the shaft (Wissmann et. al. 2001, Caskey 2001). The increase in lateral stress or horizontal pressure in the matrix soil as a result of ramming aggregate into the soil provides for the benefit of greater capacity and greater load diffusion in the matrix soil (Schmertmann 2005).

Figure 4 shows a typical 4-bar uplift RAP detail. The limit state of unit shaft resistance is computed as the unit resistance (f_s) to vertical movement as:

1. The product of the effective horizontal earth pressure ($\sigma'_h = K_p\sigma'_v$) and the tangent of the unimproved soil friction angle (ϕ'_s) for sandy soil or over-consolidated, clay soil as shown in Equation 1, where σ'_v is the effective vertical stress and K_p is the Rankine passive earth pressure coefficient (Lawton et al. 1994). The Rankine passive earth pressure coefficient is used in design up to a maximum pressure often limited to 120 to 144 kPa (2,500 to 3,000 psf). This limit is based on field measurements of lateral stress using the Ko-Stepped Blade next to installed RAPs (Handy 2001, Wissmann, 2001).

   for sand or over-consolidated clay soil: $$f_s = \sigma'_h \tan(\phi'_s) \quad (1)$$

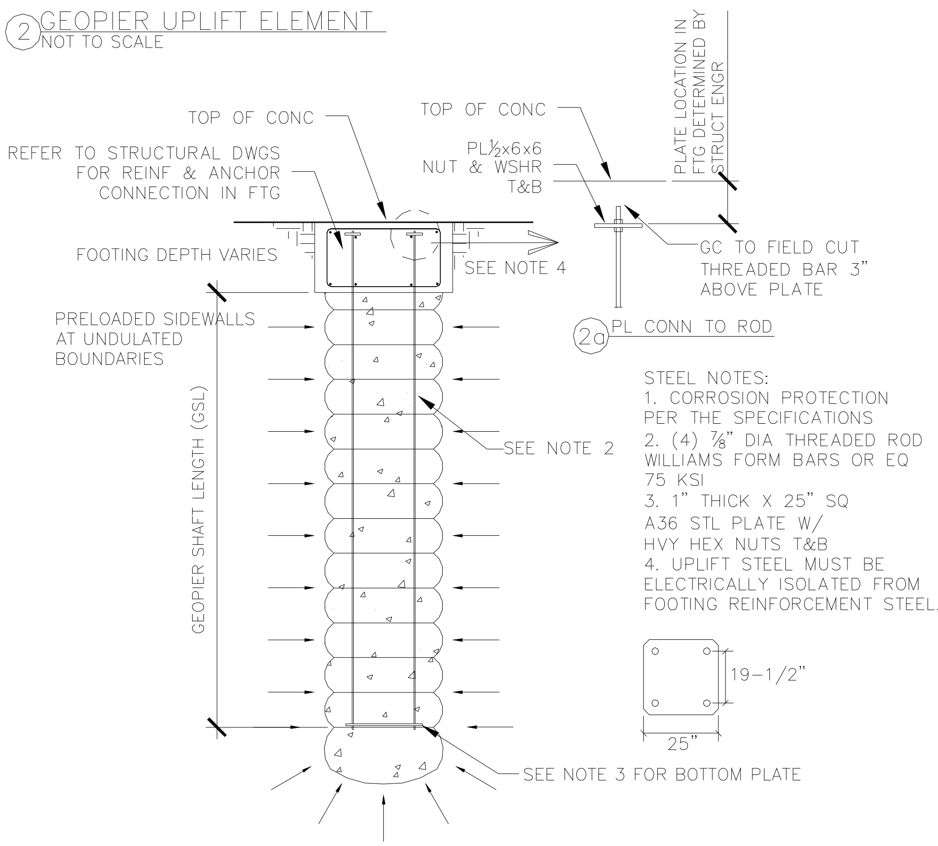


Fig.4. Typical Detail of a 4-bar RAP Uplift Element

2. The undrained shear strength (s_u) for normally-consolidated to slightly over-consolidated clay soil as shown in Equation 2. The RAP installation process results in lateral stress increases (Handy 2001). Although this increase in lateral stress causes consolidation and subsequently an increase in the undrained shear strength of the matrix soil, a conservative pre-installation estimate of undrained shear strength is used for design.

   for normally- to slightly over-consolidated clay soil: $f_s = s_u$ (2)

The ultimate uplift capacity (T_{ult}) is computed by integrating the unit uplift resistance (f_s) over the assumed cylindrical perimeter area (A_s) of the RAP plus the weight of the uplift RAP (W_{RAP}):

$$T_{ult} = f_s A_s + W_{RAP} \quad (3)$$

A factor of safety of 2 is commonly applied to the theoretical ultimate uplift capacity to assign the allowable capacity for design. Typical geotechnical uplift capacities of 222.4 to 355.8 kN (50 to 80 kips) ASD are calculated and tested in California. While an additional geotechnical limit state that addresses uplift capacity of a RAP group as described in Wissmann et al. (2001) is checked, the single pier capacity is typically the controlling consideration.

Structural Limit State Considerations

The structural design of the RAP uplift anchor must take into account these structural limit states: 1) the tensile load capacity of the anchor rods, 2) the bending of the bottom plate, and 3) anchorage of the rod in the concrete footing.

Anchor rod capacity (Q_{yield}) is calculated by either Load and Resistance Factor Design (LRFD) or Allowable Stress Design (ASD) design approach.

$$Q_{yield} = F_{yield} \sum A_{rod} \tag{4}$$

where Q_{yield} is the yield strength of the threaded rod group, F_{yield} is the yield stress, and ΣA_{rod} is the sum of the minimum cross sectional area of the threaded rod. In most environments, corrosion protection is taken into account using procedures outlined in FHWA-NHI-00-044 (Elias, 2000). Typically a design life of 75 years is considered for permanent structures. Additional width is then added to the rod diameter to enhance the corrosion protection measures described above.

For Allowable Stress Design (ASD):

$$Q < Q_A \tag{5}$$

$$\text{where } Q_A = 0.60\ Q_{yield} \tag{6}$$

where Q equals uplift demand at "working stress" levels

For Load and Resistance Factor Design (LRFD):

$$Ru < \phi Rn \tag{7}$$

$$\text{where } \phi Rn = 0.9\ Q_{yield}\text{, and} \tag{8}$$

Ru is the critical factored load based on the governing load combination

The minimum plate thickness with regard to bending of the bottom plate can be estimated with a closed-form equation. However, this tends to over-estimate the plate thickness. A finite element analysis using "thick" plates provides for an optimized solution. The typical uplift RAP application consists of an 838 mm (33 inch) drilled diameter RAP equipped with four #7 all-thread bars of 22.2 mm (7/8 inch) in diameter, 517 MPa (75 ksi) that extend 3.7 to 6.4 m (12 to 21 feet) to the bottom plate with a minimum plate thickness of 25 mm (1 inch). The plate thickness is also increased to enhance the required corrosion protection measures.

Anchorage of the rods in the footing is determined using conventional concrete procedures. To reduce embedment length in the concrete footing, 102 mm (4-inch) square steel plates are used on the anchor bars in the footing.

UPLIFT RAP LOAD TEST RESULTS

Uplift RAP elements are tested under single load cycles and multiple load cycles in general accordance with ASTM pile uplift test method D3689 Standard Test Method (1995). Cyclic testing protocols are generally non-standard for the geo-structures industry, however, these tests provide useful data to understand the RAP load deformation relationship. The following section presents the continuous load, cyclic axial uplift test results for projects in Sacramento and Pleasant Hill, California.

The results of two consecutive cyclic uplift load tests at 1801 L Street project in Sacramento, CA are shown in Fig. 6. The soil profile at this site consists of alluvial, loose sandy silt extending 9.5 m (30 feet) below the ground surface (bgs). Groundwater was encountered at about 2 m (6.5 feet) bgs. Standard penetration test (SPT) "N- values" ranged from 2 to 10. Moisture contents ranged from 24 % to 33% for soil above and below the groundwater, respectively.

The maximum applied load during the test was 534 kN (120 kips). The tests indicate an initial deflection of about 5 mm (0.2 inch) at the design load of 267 kN (60 kips). As the test continued, a maximum deflection of about 17 mm (0.67 inch) was observed after 5 load cycles under twice the design load 534 kN (120 kips). This test shows the repeatability of stiffness of the uplift RAP in tension in soft soil after two multiple cycle events. Based on the deflection data, it is observed that almost 50% of the initial elastic deflection is attributed to elongation of the steel anchor bars.

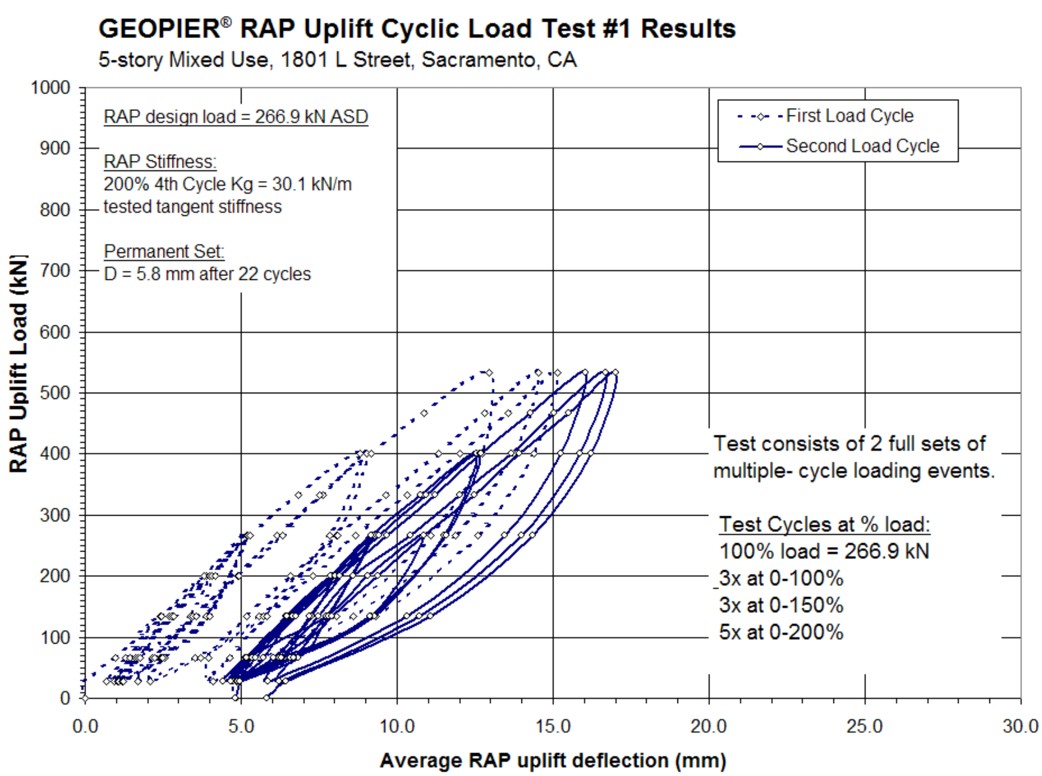


Fig. 6 Cyclic Load Test on 838 mm dia. by 4.6 m deep uplift RAP

This uplift RAP maintained linear tangent stiffness after multiple cycles performed to levels of 200% of the design load.

Cyclic uplift tests were also performed at the BART Parking Structure project located in Pleasant Hill, California. The soil profile at this site consists of alluvial soil deposits of stiff clay interbedded with medium dense sand to depths of 12.2 to 13.7 m (40 to 45 feet). Groundwater was observed at 4.6 m (15 feet) bgs. Siltstone (bedrock) was encountered at depths of 13.7 to 16.8 m (45 to 55 feet) bgs. The soil exhibited SPT N-values ranging from 6 to 10 and undrained shear strengths between 50 and 110 kPa (1044 to 2300 psf) to depths of 9.1 m (30 feet) bgs. Figure 7 shows the average undrained shear strength estimated from 3 cone penetration tests performed at the BART site.

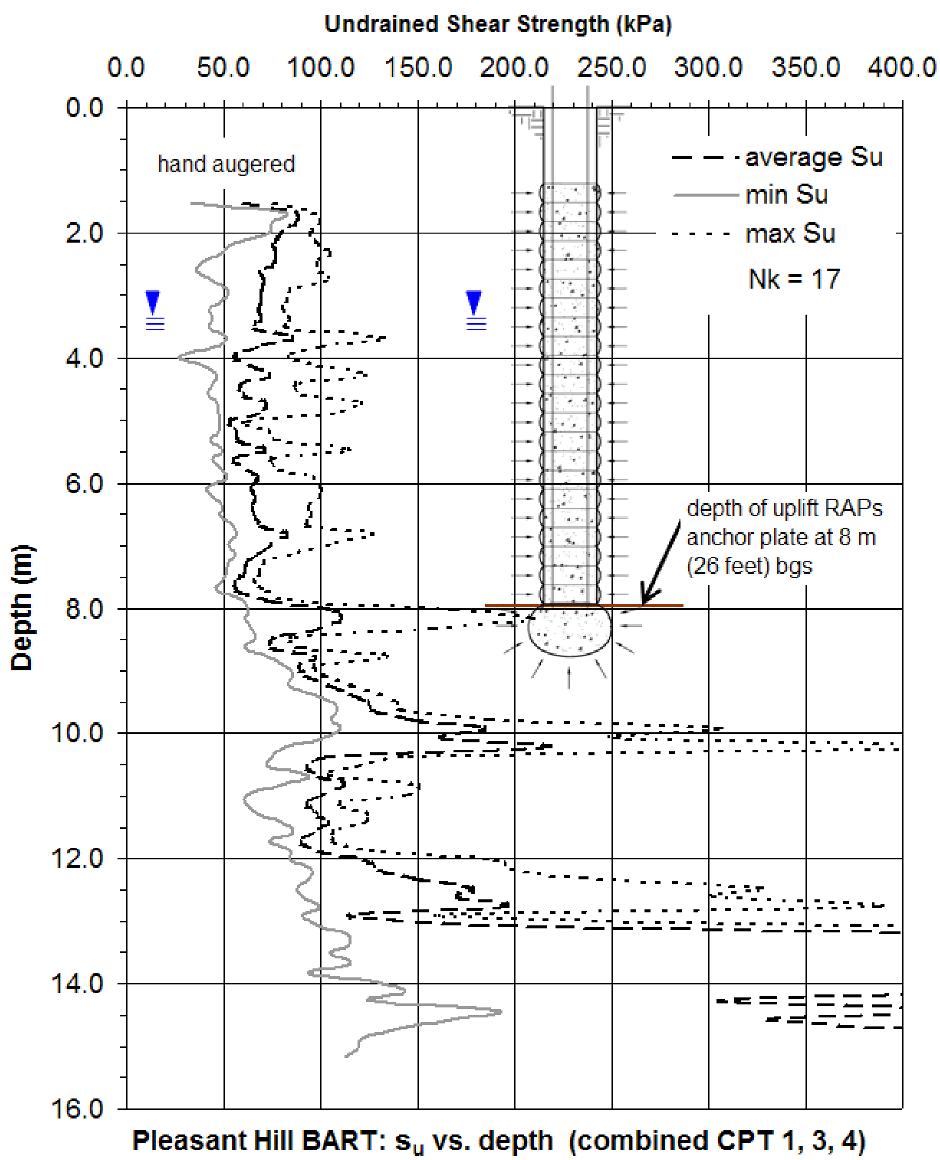


Fig. 7 Average undrained shear strength at BART Parking Structure.

The BART project required multiple full scale bearing and uplift tests at different test locations across the large site. Cyclic loading was performed on two of the uplift tests and plots of the results are shown in Figures 8 and 9. The maximum applied load during the tests was 890 kN (200 kips).

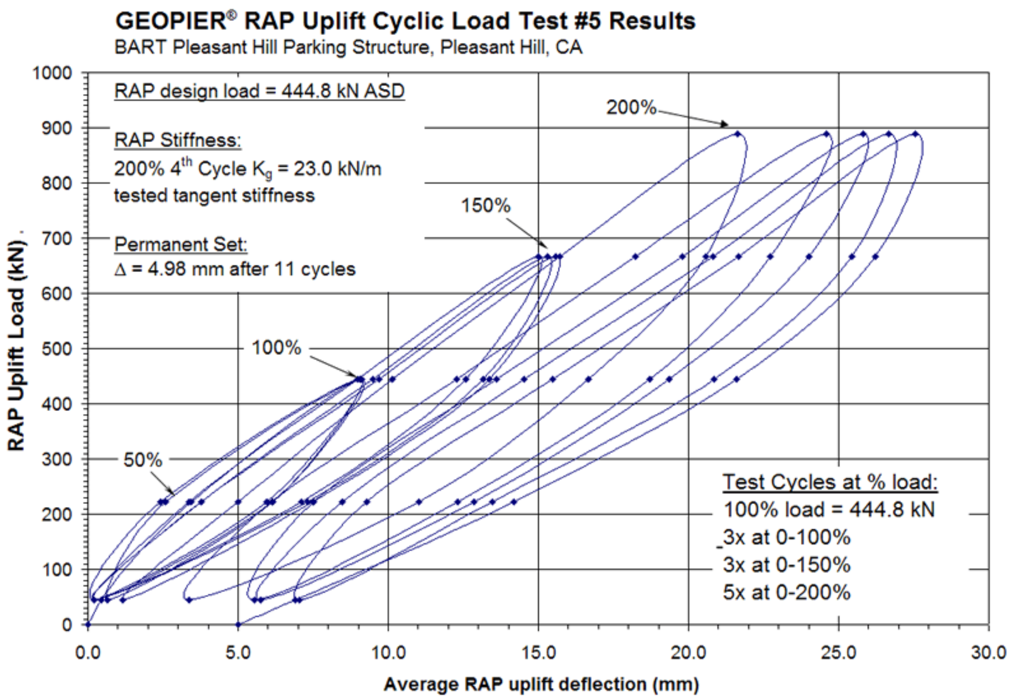


Fig. 8 Cyclic Load Test #5 on 838 mm dia. by 6.1 m deep uplift RAP

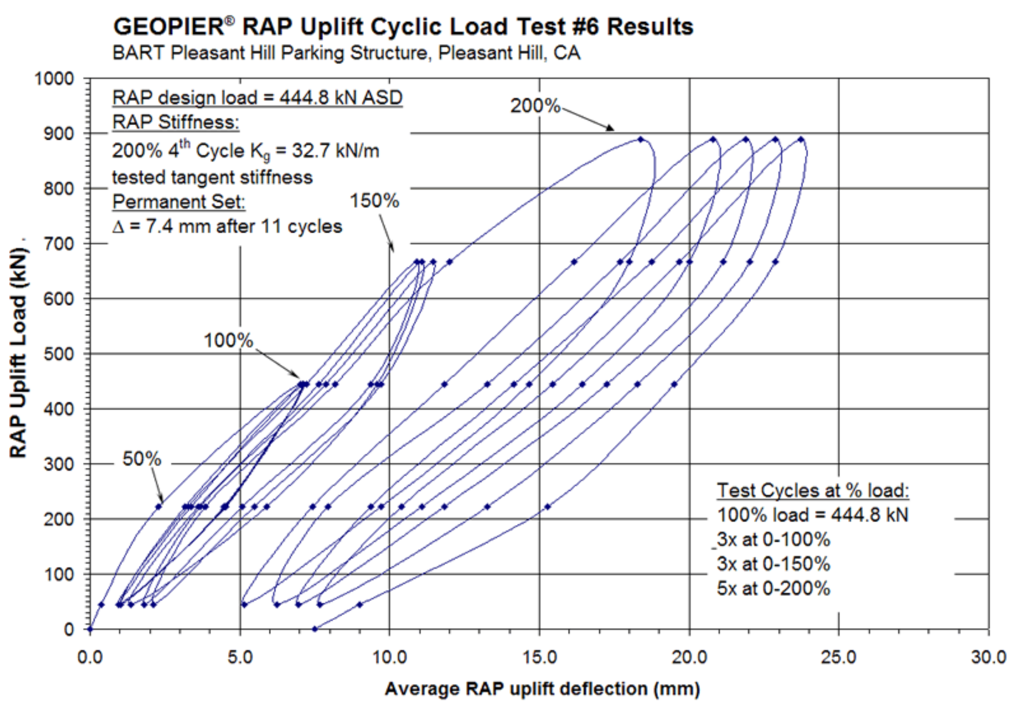


Fig. 9 Cyclic Load Test #6 on 838 mm dia. by 6.1 m deep uplift RAP

The BART test results indicate deflections of less than 10 mm (0.39 inch) at the design load of 445 kN (100 kips) and typically less than 25 mm (1 inch) of deflection after repeated cycles at 200% of the design load. Once again these tests show the repeatability of linear stiffness of between 23 to 33 kN/m (204 to 292 kips/inch) of the uplift RAP in tension in stiff clay and medium dense silty sand after multiple load cycles. Figure 10 shows a group of uplift RAPS in a shearwall mat at the BART site in Pleasant Hill.

Fig. 10 Group of uplift RAPs at BART in Pleasant Hill, CA.

SUMMARY OF UPLIFT RAP CYCLIC TENSION TESTS

Table 1 summarizes the RAP Design Parameter values and Uplift Test data for pseudo-cyclic uplift tests performed in Sacramento and Pleasant Hill, CA. The results of the data indicate the piers exhibit a relatively consistent load-deflection behavior under multiple loading cycles performed up to 200% of the allowable design capacity. Additionally, while the cyclic deflections observed during testing were up to 27 mm (1.1 inch), the permanent deflections measured after the conclusion of the cyclic testing are less than 8 mm (0.3 in). It is also interesting to observe that a significant contribution to the total deflection of the uplift RAPs at 200% design load is related to elongation of the anchor bars.

These pseudo-cyclic tests were performed as a part of research efforts to better understand the load-deformation relationship of uplift RAP elements. While the relationship between number of cycles and deformation is not specifically included as a design parameter, it is interesting to note from these test results that the stiffness does not degrade with additional load cycles. In addition, while each load cycle produces incremental plastic deformation (set) of the soil/bottom-plate zone, the increment of plastic deformation generally decreases with increased load cycles. This behavior is likely related to the cavity expansion and lateral stress increases imparted during RAP installation. This behavior also results in repeatable tangent stiffness with virtually no degradation after as many as 22 continuous load cycles.

Table 1. Summary of RAP Design Parameters and Uplift Tests

Soil and Test Parameters	1801 L Street	BART Test 5	BART Test 6
Soil Profile, m (feet)	ML to 9.5 (30)	CL to 12.2 (40)	CL to 12.2 (40)
Unit weight, kN/m^3 (1b/ft^3)	18.1 (115)	19.6 (125)	19.6 (125)
Ground Water, m (feet)	1.5 (5)	3.0 (10)	3.0 (10)
N_1 - mid depth, blows/0.3m	5	10	10
N_1 - Low/Hi, blows/0.3m	2/10	6/21	6/21
Su (Hi/Lo), kPa (psf)	0	71 (1,250)	71 (1,250)
Phi angle, degrees	34	0	0
RAP Installation Method	Replacement	Replacement	Replacement
RAP Diameter, m (feet)	0.83 (33)	0.83 (33)	0.83 (33)
Footing Depth, m (feet)	0.9 (3)	1.8 (6)	1.8 (6)
RAP Shaft Length, m (feet)	4.6 (15)	6.7 (22)	6.7 (22)
Total Drill Depth, m (feet)	5.6 (18)	5.6 (18)	5.6 (18)
Est. Capacity, kN (kip)	934 (210)	961 (216)	961 (216)
FS for Design Capacity	3.5	2	2
ASD Capacity, kN (kip)	267 (60)	445 (100)	445 (100)
Tested Capacity, kN (kip)	534 (120)	890 (200)	890 (200)
Bar Strain, mm (inch)	9.6 (0.38)	14.1 (0.56)	14.1 (0.56)
Tested RAP Deflection at 200% ASD Cap., mm (inch)	13.5 (0.53)	27.6 (1.09)	23.7 (0.93)
Actual Plate Deflection at 200% ASD Cap., mm (inch)	3.9 (0.15)	13.5 (0.53)	9.6 (0.38)
Permanent Plate Deflection at end of test, mm (inch)	5.8 (0.2)	5.0 (0.2)	7.4 (0.3)
RAP Stiffness at 200% ASD Cap., kN/m (kip/inch)	30.1 (266.4)	23.0 (203.6)	32.7 (289.4)

CONCLUSIONS

As building codes have consolidated into the IBC and with California's adoption of the 2007 California Building Code, the California version of the IBC, the need for cost-effective foundation solutions to resist high seismic foundation loads can be mitigated using Rammed Aggregate Pier solutions. In high seismic and wind regions, the resulting overturning forces are effectively resisted with uplift RAPs. RAP installation methods increase the lateral stress in the soil, thereby increasing the RAP uplift capacity at shallow depths. Full scale load test data presented herein demonstrate the performance of uplift RAP elements for the resistance of uplift forces on foundations. Typical RAP ASD uplift capacities in California range from 178 to 445 kN (40 to 100 kips) for 762 to 838 mm (30 to 33 inch), 3.6 to 7.0 m (12 to 23 feet) long RAP elements and incorporate a minimum factor of safety of 2. The load test results show repeatable tangent stiffness after multiple overstress load cycles to 200% of the allowable design load. Tangent stiffness exhibited by uplift RAPs ranged between 23 and 33 kN/m (204 to 292 kips/inch). A robust corrosion protection system incorporated in the design of uplift RAPs is also described. Coupling the demonstrated repeatable stiffness at high loads with multiple layers of corrosion protection on the uplift RAP components, designers and engineers can be assured of reliable hold-down resistance and long lasting performance.

ACKNOWLEDGMENTS

The authors appreciate the support of the practicing geotechnical engineers and structural engineers in California that have recommended and supported the use of rammed aggregate pier supported foundations. Particular thanks are made to Watry Design Inc. and Treadwell & Rollo Inc. for their support in multiple uplift tests at the Pleasant Hill BART project and Point 2 Structural Engineers and Wallace Kuhl & Associates for their support in uplift tests at the 1801 L Street project in Sacramento.

REFERENCES

ASTM Test Standards (1995)

Caskey, J.M. (2001). “Uplift Capacity of Rammed Aggregate Pier Soil Reinforcing Elements.” University of Memphis.

Elias, Victor (2000). “Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes”, Publication FHWA-NHI-00-044

Geotechnical Report BART Pleasant Hill, Treadwell & Rollo Inc. (2004)

Geotechnical Report 1801 L Street, Wallace Kuhl & Associates (2000)

Handy, R.L. (2001). “Does lateral stress really influence settlement?” Journal of Geotechnical and Geoenvironmental Engineering. 127(7), 623-626.

Lawton, E. C. (2000). “Performance of Geopier Foundations During Simulated Seismic Tests at South Temple Bridge on Interstate 15, Salt Lake City, Utah.” *Final Report, No. UUCVEEN 00-03*, University of Utah, Salt Lake City, Utah.

Lawton E. C. and N. S. Fox. (1994). "Settlement of structures supported on marginal or inadequate soils stiffened with short aggregate piers." *Vertical and Horizontal Deformations of Foundations and Embankments,* A.T. Yeung and G.Y. Fello (Editors), American Society of Civil Engineers, 2, 962-74.

Lawton, E.C., Fox, N. S., and R. L. Handy. (1994). "Control of settlement and uplift of structures using short aggregate piers." *In-situ Deep Soil Improvement*, K.M. Rollins (Editor), American Society of Civil Engineers, 121-132.

Majchrzak, M., Lew, M., Sorensen, K., and Farrell, T. (2004). "Settlement of Shallow Foundations Constructed Over Reinforced Soil: Design Estimates vs. Measurements." Fifth International Conference on Case Histories in Geotechnical Engineering, April 2004.

Robertson, PK, Campanella, RG, Gillespie, D, Greig, J, (1986). "Use of Piezometer Cone Data." Proceedings of inSitu 86, ASCE Specialty Conference, Blacksburg, VA

Schmertmann, J.H. (2005). "Stress Diffusion Experiment in Sand." Journal of Geotechnical and Geoenvironmental Engineering. 131(1), 1-10.

White, D.J., Wissmann, K.J., Barnes, A.G., and Gaul, A.J. (2002). "Embankment Support: A Comparison of Stone Column and Rammed Aggregate Pier Soil Reinforcement." Presented, Transportation Research Board. 81st Meeting, Washington, D.C. January 13-17.

Wissmann, K.J., J.M. Caskey, and B.T. FitzPatrick. (2001). "Geopier® Uplift Resistance." Technical Bulletin No. 3. Geopier® Foundation Company, Inc., Scottsdale, AZ.