RAMMED AGGREGATE PIER SYSTEM PROVIDES UPLIFT RESISTANCE AT UNIVERSITY ICE ARENA

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The Allwell Center Ice Arena at Plymouth State University, Plymouth, New Hampshire is located in the flood plain of the Pemigewasset River. To maintain the arena near the level of the surrounding area, foundation design required use of uplift elements to resist hydrostatic loads caused by frequent flood conditions. Soils beneath the arena consisted of very loose to medium dense alluvial deposits overlying thick glaciolacustrine deposits. Bedrock was encountered at depths of 115 feet (35 m) or more.

A mat foundation with a sealed building system was developed for the arena to guard against flood events. Several alternatives were investigated to provide uplift capacity for the mat foundation, including the Rammed Aggregate Pier® (RAP) system, driven precast-prestressed concrete piles, driven H-piles, and deep drilled soil/rock anchors. Evaluations indicated that the most cost effective uplift solution was a tension RAP system that developed uplift resistance in the shallow alluvial deposits.

A total of 344 55-kip (245-kN) capacity RAP elements were required for the project. The RAPs were designed to resist tensile loads using threaded steel bars. Dual corrosion resistance was provided to the uplift elements by including sacrificial steel and galvanizing the oversized bars and connection plates. A pre-construction test program was conducted on RAP elements drilled to a depth of approximately 25 feet (7.6 m) and installed using various shaft construction procedures and materials. Ten test piers were constructed and six were tested to verify that the piers could resist 200 percent of the design load with measured deflections of less than 1 inch (25 mm). The RAP system also provided improved bearing conditions for the perimeter wall and mat foundations by densifying and stiffening the upper alluvial soils.

BACKGROUND

The Allwell Center Ice Arena consists of a high bay, 140 foot by 315 foot (43 m by 96 m) arena recently constructed on the Plymouth State University campus in Plymouth, New Hampshire. The proposed arena site was located in an existing parking lot to the west of the University's Facilities Services Building on Bridge Street/Route 175A (see Figure 1). The existing ground surface in the proposed building area ranged from approximately El. 470 to El. 474.

Figure 1 - Site Location

The site is located within the flood plain of the adjacent Pemigewasset River, in an area plagued by frequent flood events. The 100-year flood level selected for project design was El. 485. Several alternative building and site geometries were considered by the Civil and Structural Engineer for the proposed arena in view of flood conditions. These alternates considered raising the grades of the entire arena so that flood events would not impact the integrity of the structure.

To maintain the arena near the level of the surrounding area, Level 1 (ice rink and locker rooms) was proposed to be finished at El. 477; 8 feet (2.4 m) below the design flood level. The seating area steps up to Level 2 at El. 490. The planned grades outside the building varied from El. 473 on the east side to El. 489 on the west side.

The structure was designed to be supported primarily on a reinforced concrete structural mat foundation with columns bearing directly on the mat. Perimeter foundation (retaining) walls, supported on spread footings, were required on the west and north sides of the building due to adjacent elevated exterior grading. The building was "sealed" to protect against the 100-year flood and designed for the corresponding hydrostatic pressures. Hydrostatic uplift pressures on the sealed mat foundation would need to be resisted by a combination of the mat weight and tie-down elements.

The project team included the University System of New Hampshire (Project Owner), Sasaki Associates, Inc. (Architect), and Rist-Frost Shumway Engineering, P.C. (Civil and Structural Engineer). Haley & Aldrich, Inc. provided geotechnical engineering services for the project. As described below, Helical Drilling, Inc. and Design/Build Geotechnical, LLC designed, tested, and installed the Rammed Aggregate Piers® (RAPs) used to resist uplift loads.

SUBSURFACE INVESTIGATIONS AND CONDITIONS

A geotechnical field investigation was undertaken at the arena site to determine subsurface conditions for foundation design. A total of eleven test borings were drilled and one groundwater observation well (at test boring HA9) was installed (see Figure 2).

Figure 2 - Subsurface Exploration Program

The subsurface explorations revealed that the site is underlain a relatively thin (less than 6 feet [1.8 m] thick) layer of loose to dense granular fill at the ground surface. Beneath the fill, naturally deposited alluvial deposits were encountered, ranging in thickness from 12 to 24 feet (3.7 to 7.3 m), and consisting of very lose to medium dense poorly graded sand or silty sand. The alluvial deposits were underlain by a thick layer of glaciolacustrine deposits (90 to more than 105 feet thick [27 m to 32 m]), consisting of very loose to very dense (at depth) silt and sand (see Figure 3). Bedrock was encountered at depths of 115 feet (35 m) or more.

Water levels during or shortly after drilling were estimated to be from 10 feet to 17 feet (3.0 m to 5.2 m) below ground surface in the completed boreholes, corresponding to about El. 455 to El. 461.

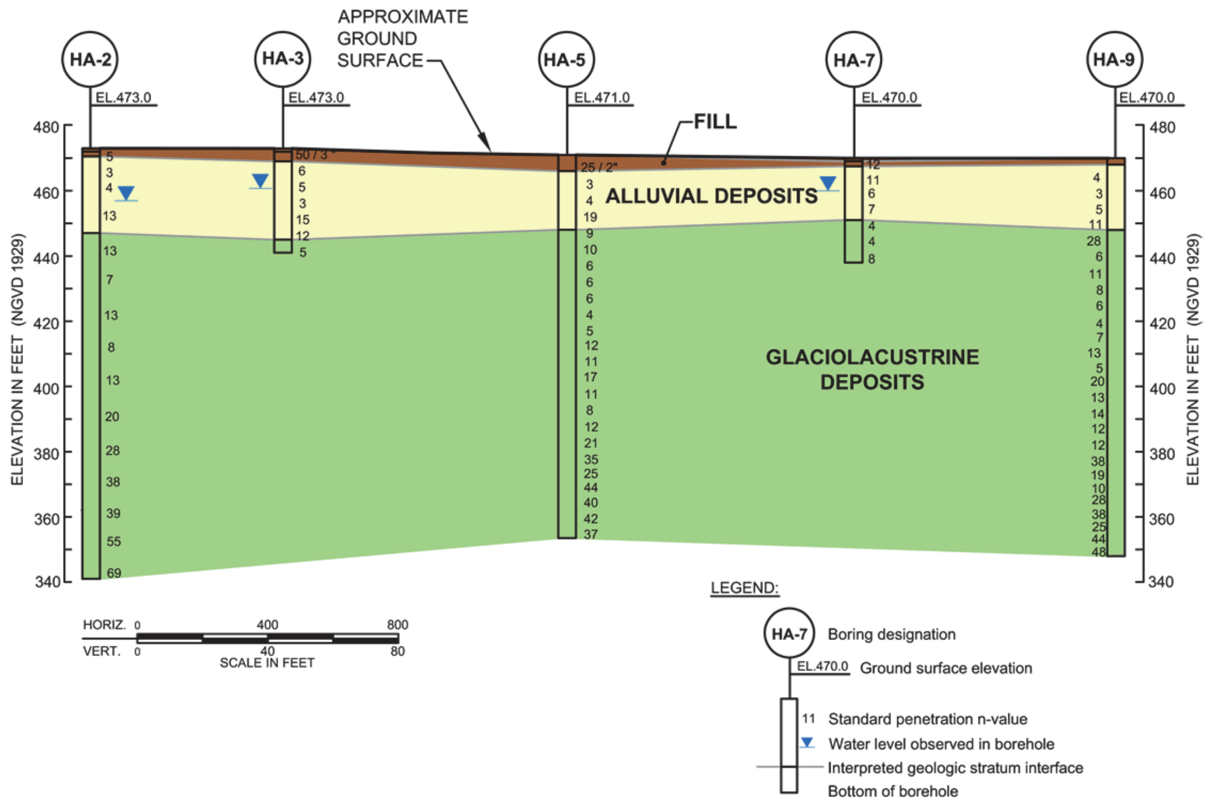

Figure 3 – North-South Geologic Section (East)

These subsurface conditions presented several challenges to the foundation design and selection of uplift elements to resist hydrostatic pressures. Both the fill and alluvial deposits were non-uniform in density, with Standard Penetration Resistance N-values values ranging from 2 to 28. These variable conditions provided non-uniform support to the mat foundation. The loose condition of the underlying glaciolacustrine deposits and the considerable depth to bedrock (115 feet or more [35 m]) significantly impacted the design and the potential cost of the uplift elements.

FOUNDATION DESIGN ASSESSMENT

The two critical geotechnical aspects of the mat foundation geotechnical design assessment included: uniform bearing of the mat foundation and hydrostatic uplift tie-down elements. These design elements are discussed below.

Uniform Bearing of the Mat Foundation

A thickened and structurally reinforced mat foundation would generally tend to level out non-uniform bearing conditions. The upper alluvial sands varied in density from very loose to medium dense and the lower glaciolacustrine deposits varied in density from very loose grading to very dense with increased depth. If the mat were subjected to very high differential settlements, excessive bending moments in the mat would occur and it may crack, thus a very thick slab would be required. In order to limit the thickness of the slab and provide for more uniform bearing conditions soil improvement would be required.

Hydrostatic Uplift

To address the hydrostatic uplift of the mat the first option that was investigated by the project team was a thickened gravity mat. Based on the required thickness of the mat, this option was dismissed early on as cost prohibitive. Several deep and intermediate depth tie-down systems were assessed. These systems included: driven precast prestressed concrete piles, driven H-piles, deep drilled soil anchors, and RAPs. “Mechanical” systems including helical anchors and Manta Ray anchors (similar to a soil “toggle bolt”) were also investigated as options but it was determined that they would not reliably provide the capacity or the long term stiffness required for this application. The systems investigated are briefly described below:

Driven Precast Concrete Piles (12 inch [305 mm] square concrete piles installed to a depth of about 55 feet [16.8 m] beneath existing grades): This option would provide for about 50 kips (222-kN) of uplift capacity per element. The system would improve the subsoils by providing some densification. Thus, it does address and improve the non-uniform bearing conditions. The cost of the system was estimated to be on the order of $1,500,000.

Driven H-Piles (HP12 x 42 piles installed to a depth of about 71 feet [21.6 m] beneath existing grades): This option provides about 50 kips (222-kN) of uplift capacity per element. H-piles are generally considered to be non-displacement piles and would do little to improve the non uniform bearing conditions and additional soil improvement would be required. The cost of the system was estimated to be on the order of $1,300,000.

Drilled Deep Soil Anchors (drilled to a depth of about 110 feet [33.5 m], into glacial till or near the top of bedrock): Higher uplift capacities can be used with this option, on the order of 150 kips (667-kN) per element. Again this system would need to be used in conjunction with a soil improvement program. The cost of the system was estimated to be on the order of $1,500,000.

Rammed Aggregate Piers (about 24 inch to 36 inch diameter [61 cm to 91 cm] and installed to a depth of about 25 to 30 feet [7.6 to 9.1 m] beneath existing grades – mandrel driven): The uplift capacity of each element was found to be about 50 kips (222-kN). The RAP option would also serve to provide a more uniform bearing surface. The cost of the RAPs was found to be about 1/3 the cost of the other options that were investigated.

Based on the above assessments, mandrel driven RAPs were selected as the tie-down elements in view of the hydrostatic uplift requirements and the need for soil improvement. A total of 344 55-kip (245-kN) capacity tie-down elements were required to resist the excess hydrostatic uplift.

SPECIFIED REQUIREMENTS OF THE RAMMED AGGREGATE PIER FOUNDATIONS

The primary purpose of the RAP system is to provide uplift support for the sealed mat foundation. The RAP system also needs to provide a more uniform bearing condition for the mat foundation. Recommended design criteria for RAPs were determined to be:

- RAPs should be installed at a uniform pre-determined spacing(s) beneath the mat to provide the design uplift capacity. Initial design could assume a 50-kip (222-kN) uplift capacity for RAPs installed at 10 to 12 foot (3.0 m to 3.7 m) center-to-center spacing.

- Detailed design of the RAP configuration and installation, including a field load testing/ confirmation program, should be provided by the RAP design/build specialty contractor, with design review and field monitoring by the project Geotechnical Engineer.

- RAPs should be tested to verify the design uplift capacity. A series of uplift tests (minimum of two) should be performed to a test load of at least 200 percent of the uplift design capacity.

- Design of the steel anchor plates and tie rods must provide permanent protection against corrosion. The design should incorporate galvanization, sacrificial perimeter steel (at least 1/16-inch [0.16 cm] perimeter thickness) and/or other measures as appropriate to provide redundant, permanent protection.

- RAPs should densify the alluvial soils. These materials were identified to be about 15 feet (4.6 m) below the bottom of the mat foundations.

- RAP construction needs to consider the presence of relatively shallow groundwater.

- The connection between the RAP uplift element and the structural slab will need to be coordinated with the project Structural Engineer.

RAMMED AGGREGATE PIER UPLIFT ELEMENT DESIGN

The use of RAPs to resist uplift loads has become more common in recent years for several reasons. This is partially related to increased seismic code requirements making uplift more common and part due to advances/experience in installation techniques and experimental testing of RAP uplift elements which demonstrated significant resistance capabilities. RAP uplifts have become another reliable tool for the structural and geotechnical engineer to consider in the design/cost estimating process.

As with any design, there are specifications (capacity and deflection) that need to be met and experimental testing over recent years has lead to the design that Helical Drilling, Inc. (HDI) currently employs. The capacity of an uplift RAP is predominantly driven by the soil profile and RAP diameter assuming the structural design is sufficient. For any given soil profile a larger diameter RAP will provide more uplift resistance than a smaller diameter RAP. The RAP diameter is a function of the mandrel diameter used to construct the pier and the construction sequence building the RAP (see Figure 4).

Figure 4 - Rig and Mandrel

The RAP construction sequence consists of driving the mandrel to a pre-determined depth based on calculations and design considerations and then building the RAP. Building the RAP from the bottom up is achieved by raising the mandrel 4 feet (1.2 m) and re-driving 3 feet (0.9 m) (called a 4/3 stroke for example) to construct the RAP one lift at a time. This process is continued up to the ground surface.

The constructed RAP diameter is determined based on the stone volumes used during pier installation and largely depends on the soil profile and construction process.

As discussed above, the RAP deflection under load is predominantly a factor of the soil profile but also a factor of the structural make-up of the uplift "harness." Experience has shown that deflection measured at the ground surface related to the harness design can be minimized by a robust harness (see Figure 5) and grouting the lower portion of the uplift RAP (see Figure 6).

Figure 5 - Tendon and Baseplate

Figure 6 - Grouted RAP

The harness bottom plate should be thick enough to resist bending under uplift load applied to the hold down tendon rods. Additionally, it has been found that grouting the bottom portion of the RAP stiffens the harness assembly thereby reducing any bending of the bottom plate which could cause additional deflection at the ground surface. The tendon rods should be of sufficient area to minimize PL/AE elongation. The plate thickness and tendon diameter are project specific based on load requirements. Plates 0.5 in. (1.3 cm) to 1.5 in. (3.8 cm) and tendons size six to ten bar are common. Specific to this project, double corrosion protection was required. Galvanization and sacrificial steel was proposed by HDI and accepted by the project team.

Test Program

For this size project HDI and Haley & Aldrich determined that a pre-construction test program should be conducted to verify design uplift calculations and determine the optimal installation construction sequence to meet the project specifications. It should be noted that group action of the RAPs was considered and checked but it was determined that RAPs act independently of one another. In order to achieve the required uplift capacity it was decided to extend piers into the glaciolacustrine deposits below the sands for a total depth of 26 to 28 feet (7.9 to 8.5 m) below ground surface. The pier diameter was calculated to be 25 in. (64 cm). The ultimate load (pull out of the RAP) was calculated to be 180 kips (801 kN) for this configuration.

The location of the test program was chosen based on borings to the north which indicated some of the poorer soil conditions at the site and a location that was accessible at the time the test was to be completed (see Figure 7).

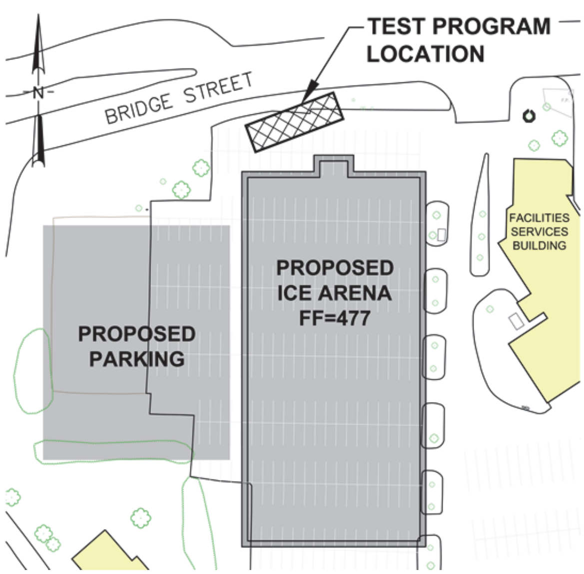

Figure 7 - Test Program Location

A total of ten test RAPs were installed and six were ultimately tested.

The upload test set-up is shown on Figure 8. The uplift tests were performed in general accordance with ASTM 3689, quick load test method.


Figure 8 – Load Test Set-Up

For this specific site and the variables involved, the test program needed to demonstrate which construction techniques would most effectively attain the required capacities. The techniques that were varied in the test program were the mandrel stroke pattern and the grouted length of the RAP (see Table 1).

Table 2 presents a summary of the test pier performance data. Figure 9 is a summary plot of Deflection versus Load plot comparison of the pier stiffness with varying grout lengths. Note that the fully grouted pier has only slightly less deflection at the design load than a pier with a third of its length grouted. Figure 10 is a Deflection versus Load plot comparison with the stroke pattern chosen. This plot indicates that the increased stroke has only a slight increase in the RAP stiffness (less deflection) at the design load.

Table 1 - Pier Construction Details

Test Pier Number	Telltale Depth (ft)	Stroke Pull/Push (ft)	Pier Volume (ft^3)	Nominal Diameter (in)	Grout Volume (ft^3)	Grouted Shaft (ft)	Remarks
TP-1	26.2	3/2	91.4	25.3	3.4	8.4	*
TP-2	26.0	3/2	88.4	25.0	3.4	8.4	*
TP-3	27.5	4/3	82.1	23.4	3.4	8.4	*
TP-4	28.0	4/3	87.2	23.9	3.4	8.4	*
TP-5	28.5	5/4	114.3	27.1	3.4	8.4	*4/3 stroke in grout zone
TP-6	26.0	5/4	112.3	28.1	3.4	8.4	*4/3 stroke in grout zone
TP-7	26.0	3/2	85.6	24.6	10.1	25.2	*Fully grouted
TP-8	27.5	3/2	90.7	24.6	10.1	25.2	
TP-9	26.0	3/2	89.6	25.1	3.4	8.4	3/1 stroke for 1st lift
TP-10	26.0	3/2	86.4	24.7	3.4	8.4	3/1 stroke for 1st lift

* tested piers

Table 2 - Summary of Test Pier Performance Data

Test Pier Number	Deflection at 100% Load (in)	Deflection at 200% Load (in)	Creep 117% Load	Remarks
TP-1	0.32	1.00	0.01 in./ 60 min.	3/2 stroke, Jack failed at 150% load increment, pier reloaded.
TP-2	0.24	0.77	0.00 in./ 60 min.	3/2 Stroke
TP-3	0.20	0.60	0.00 in./ 60 min.	4/3 Stroke
TP-4	0.20	0.64	0.01 in./ 70 min.	4/3 Stroke
TP-5	0.24	1.15	0.01 in./ 60 min.	5/4 Stroke, 4/3 in Grout zone, Failed attempting to reach 225% on reload cycle
TP-7	0.21	0.60	0.01 in./ 60 min.	Fully grouted

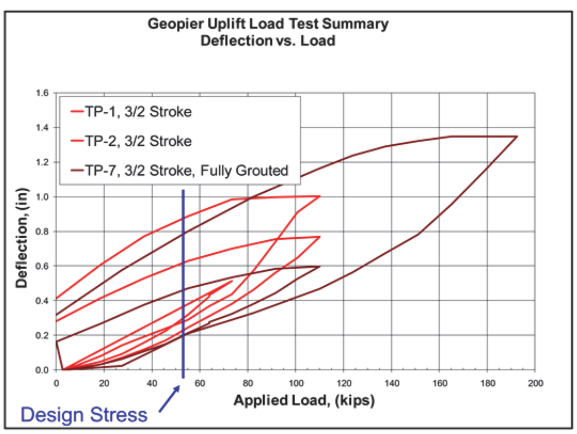

Figure 9 - Influence of Grout Length

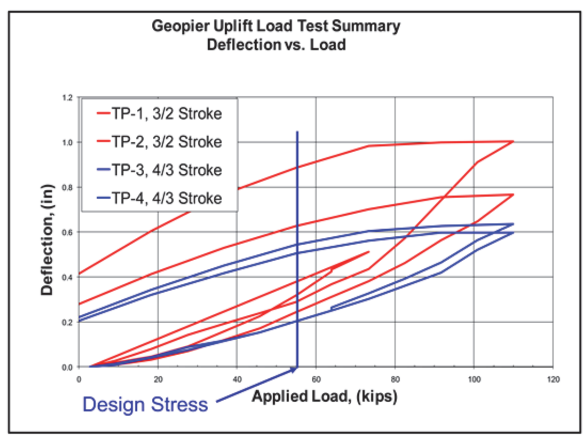

Figure 10 - Influence of Stroke Pattern

All tests demonstrated compliance with the project specifications of 1/2 inch (1.3 cm) of deflection at the design load and 1 in. (2.5 cm) or less of deflection at 200 percent of the design load.

Results

As a result of the test program, Haley & Aldrich and HDI concluded that production RAP's should be constructed with a 4/3 stroke and a minimum grout length of 8 feet (2.4 m).

A total of 344 production RAPs were successfully installed after the testing program. The RAP's were structurally tied to the mat foundation.

CONCLUSIONS

RAP foundations were selected for the Plymouth State Ice Arena because this foundation type could: 1) improve the near surface conditions by providing uniform bearing conditions; 2) economically provide uplift resistance to a sealed mat foundation that is subjected to flood conditions. The more extensive field test program showed that the RAP analytical methodology was confirmed for these subsurface conditions and the RAPs had sufficient capacity to resist the imposed uplift loading.

REFERENCES

GEOPIER FOUNDATION COMPANY, INC., 2001. Technical Bulletin No. 3 – Geopier Uplift Resistance, pp. 1-11.