

# Micropile Underpinning of a Machine Foundation

by Allen W. Cadden, Donald A. Bruce, and Robert Traylor

The production facility of a plastic injection molding equipment company was constructed in 1989 and is supported on shallow spread footings with a 10 in. (250 mm) thick floor slab-on-grade. Equipment developed and tested at this facility has static weights of up to 1,000,000 lb (4500 kN). Some of the machinery has moving components that result in about 25 percent of the load being transient.

As development of the machinery has progressed over the years, its size and loading have increased, leading to unacceptable deflections detected during the assembly of a large unit. Such deflections cause rapid wear of precision glide plates and tiebars — critical elements in the performance of the equipment.

The machine is supported by a rectangular steel frame (Fig. 1) consisting of a built up steel I-beam with web stiffeners. The frame is supported on the floor slab by adjustable bearing pads measuring 8 x 10 in. (200 x 250 mm) and spaced on 24 in. (610 mm) centers along each side of the bottom flange of the I-beams. The static weight of the equipment is about 900 kips (4000 kN), of which about 250

kip (1100 kN) are supported within a clamp assembly near the center of the machine [the clamp travels along the tiebars for about 100 in. (2500 mm)].

During initial testing of the machine, distortions were measured at the adjustable bearing pads, and minor settlements occurred in the north end of the clamp base, thus generating distortion in the northwest direction. Furthermore, the equipment experienced "rocking" during initial test runs. The settlements, distortions, and rocking hampered the testing of the equipment and caused wear of the brass glide plates. The alignment of the centerline of the two halves of the machine was also disrupted.

A 3-week test cycle was required prior to dismantling and shipping the machine. Due to the difficulties and time involved with dismantling, moving, and re-erection, foundation underpinning was investigated to limit the movements of the equipment to angular distortions of less than 0.005 in. (0.13 mm) over 10 ft (3.0 m) during operation, and differential settlement between the

two halves of the machine to essentially zero.

## Site and geotechnical conditions

The facility consists of a large steel frame structure with clear overhead heights of nearly 40 ft (12 m). Within the structure, components are assembled in an area where overhead gantry cranes assist in the handling of larger components and finished products.

The assembly area is accessible through several roll top doors at each end of the plant where tractor-trailers enter the building for loading. Given the plant access conditions, field exploration was performed utilizing truck-mounted and articulated frame-mounted drill rigs to core through the floor slab and perform standard test borings with continuous sampling from the slab level to the surface of the

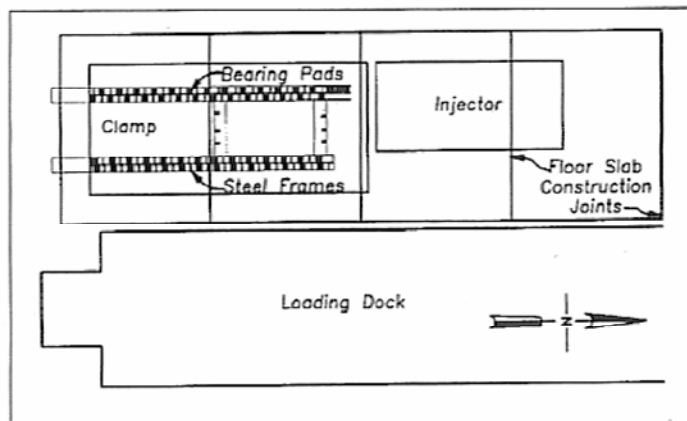


Fig. 1 — Overall site plan of machinery layout showing support frame and bearing pads.

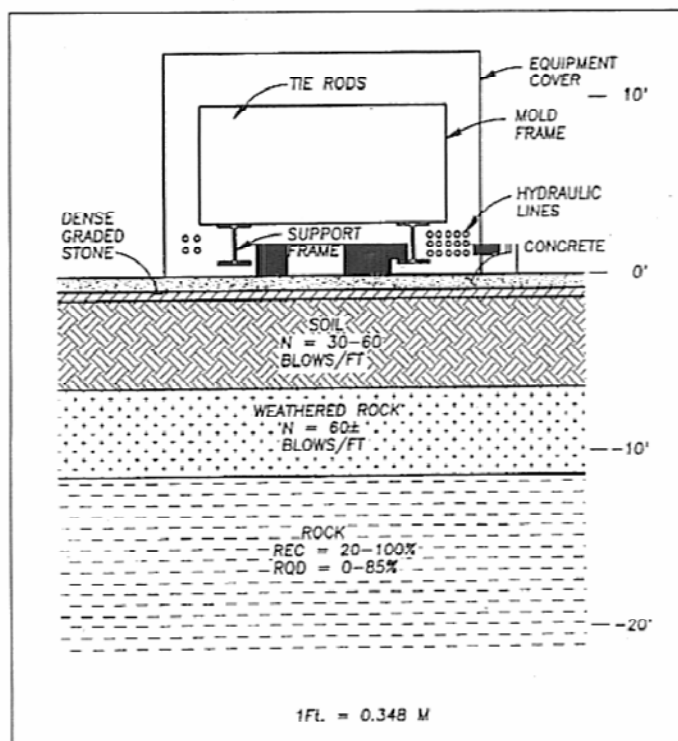


Fig. 2 — Simplified geologic layers beneath machinery.

rock. High speed diamond core drilling was also used to gather information on the underlying bedrock type and strength. Due to access limitations for the truck-mounted equipment around the machinery, hand sampling was also performed in several areas to better define the consistency of the near-surface soils.

The floor slab thickness in the areas explored ranged from 8.5 to 10 in. (210 to 250 mm) and contained welded wire reinforcement near the bottom of the slab. Just below the floor slab, dense graded crushed stone was present — the stone bedding was about 3 to 8 in. thick (76 to 203 mm). The subsurface data collected close to the machinery indicated the presence of medium-stiff to hard natural silts and clays with relatively minor amounts of sand present beneath the stone to depths of about 3 to 8 ft (1.0 to 2.5 m) (Fig. 2). Below this material the soil was very stiff with standard penetration *N* values in excess of 60 blows per ft (300 mm).

Underlying shale was encountered at depths of about 9 to 16 ft (3 to 5 m), and showed low rock quality designation values typical of the thin bedding characterizing the local geology. Measured unconfined compressive strengths were around 14,000 psi (97 MPa). Groundwater was encountered in only one of the borings and was believed to be associated with water trapped within the stone backfill of a nearby loading dock wall. Although arrangements were made to perform the site exploration during regular work hours, a stipulation was made that the remedial construction work had to be performed between 3:00 p.m. and 6:00 a.m. and on weekends, since full plant operations had to be maintained.

## Solution concepts

Exploration data confirmed that the soils were well-suited for the support of relatively heavily loaded floors or modestly loaded spread footings — given the competent soil conditions and the modest depths to rock, bearing capacity and settlements were not expected to be a problem for typical floor or foundation loading conditions. This conclusion was further supported by past satisfactory performances of machinery in the area.

The new machine, however, had

transferred excessively high loadings to the floor slabs at the bearing pads, a significant portion of which was transient. Excessive distortions resulted from this combination of structural and subsurface conditions. First, the underlying soils (particularly materials within 2 to 3 ft [0.6 to 1.0 m] of the bottom of the slab) were believed to have reached two critical states. The apparent preconsolidation or maximum past pressure may have been exceeded, resulting in a disproportionate increase in the magnitude of settlement compared to the past loading.

***“Given the subsurface conditions, limited access, and the request to perform the work with the machine in place, the remediation techniques developed had to be adaptable to the conditions encountered during construction.”***

Another condition in the soil that may have occurred was elastic deflection under the transient load. The irregular rocking observed was likely due to the variations in the soil stiffness modulus values (different soil strengths) within the shallow depths below the slab — the result of natural soil variations. This irregular soil stiffness may have also been the result of disturbance during construction and nonuniform compaction of the material just below the floor slab.

Secondly, the relatively thin and unreinforced nature of the slab resulted in the floor performing as a thin flexible member rather than as a rigid foundation system. The slab underwent both elastic deflections as the transient loading occurred, and permanent deformation from the high dead loads. In addition, the foundation slabs consisted of three isolated units, each about 20 x 25 ft (6.1 x 7.6 m). Without the benefit of continuous reinforcement and a uniform continuous mat beneath the entire area, no transfer or spreading of the loading from one area to another occurred, permitting rocking and differential settlement. It is interesting to note that no cracking of the floor slab was observed.

Since the subsurface investigation did not reveal excessively weak soils or poor concrete quality, the remedial concepts focused on increasing the composite stiffness of the foundation slab-soil system. Keeping in mind that the minimal strength of the slab precluded direct transfer of significant

loads to the underlying rock, the remedial concepts had to limit concentrations on either the floor or the machine frame, which could result in twisting the frame.

Given the subsurface conditions, limited access, and the request to perform the work with the machine in place, the remediation techniques developed had to be adaptable to the conditions encountered during construction. The “do nothing” alternative was not a feasible option since the equipment had to complete a test cycle before it could be shipped. Like-

wise, the alternative to construct a new foundation in another part of the plant and move the machine would far exceed the allowable time frame for delivery.

Past experience has proven that one of the most cost-effective measures to increase the load-carrying characteristics of certain types of soils is compaction grouting (Warner, 1973). However, this process of injecting a low mobility grout to densify and stiffen the soil would not have been effective given the nature of the soil at the site. This was further confirmed with a full-scale field test during construction. Due to the soil conditions, other types of grouting, such as permeation, compensation, or jet grouting, would not meet the project needs due to their ineffectiveness in the soils and loading conditions, or to the disturbance they would cause to plant operations (as was the case for jet grouting).

Some consideration was also given to post-tensioning the slab to minimize movements by tying it down. However, the strength of the slab was not sufficient to resist heavy concentrated loads. In addition, the soil types would likely continue to consolidate and release the post-tension load, allowing the rocking to resume. Conventional deep foundation systems would not have been constructable without removing the equipment and constructing pile caps or grade beams to support the frame.

These considerations led the design team in the direction of direct under-

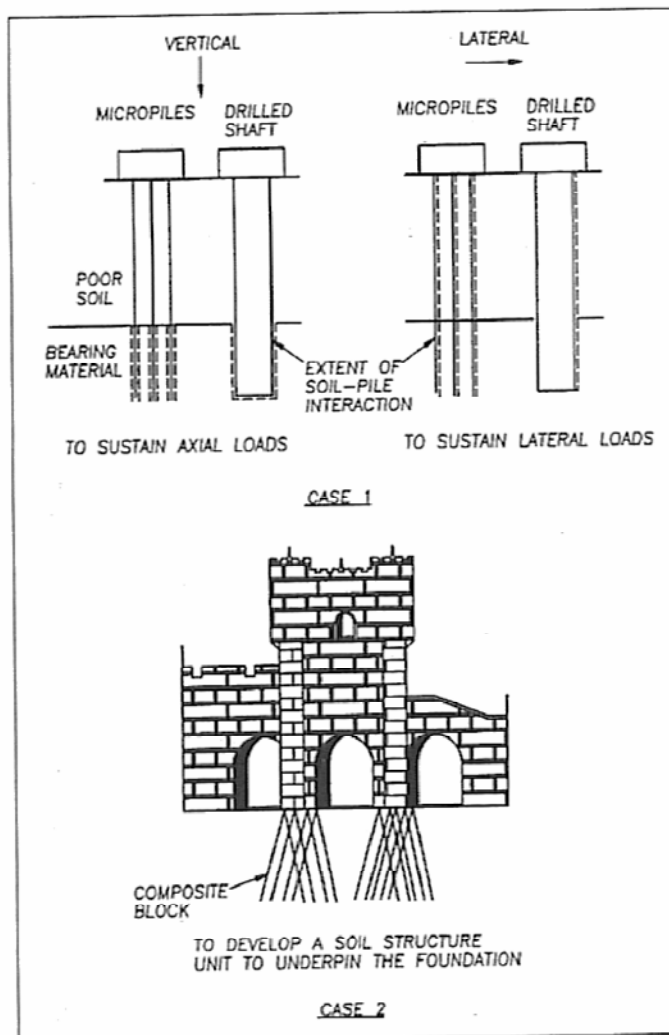


Fig. 3(a) — Classification based on design philosophy.

pinning. Conventional underpinning would require accessing the main support frame throughout its entire length (so that concentrated loads would not result in distortion of the slab and frame). The configuration of the equipment dictated that conventional pit excavation would have to be very large to reach the area beneath the frame and thus, would be too time consuming. By addressing the underpinning option in two phases — first, to improve the overall foundation system stiffness, and second, to shed some load to the underlying rock — a combination of Case 1 and Case 2 micropiles (Bruce et al., 1995) met the challenge.

### Design considerations

With respect to philosophy of design, Bruce et al. (1995) define Case 1 micropiles as those directly carrying a concentrated load and transferring it through friction to the bearing stratum. Case 2 micropiles are those installed as part of an interlocking three-dimen-

sional network that defines and internally reinforces the underlying soil mass attached to the structure. Thus, the load in a Case 2 system is carried by a stiffened and strengthened mass foundation, and not directly on individual piles or pile groups [Fig. 3(a)].

Micropiles are further classified by the grouting method [Fig. 3(b)]. Type A piles consist of those grouted under gravity head only. Type B piles are grouted with a neat cement grout at pressures typically ranging from 50 to 150 psi (0.4 to 1.0 MPa) as the casing is withdrawn. Type C is a method generally performed only in France where the hole is gravity filled with grout as the casing is withdrawn. Before the grout sets, one additional grouting is performed through a sleeve pipe at pressures in excess of 150 psi.

Type D consists of first filling the hole during casing withdrawal, as with Type A, and then, once the grout has set for several hours, post-grouting it through sleeve pipes utilizing packers to isolate the injection area. This post-grouting can be performed several times to improve the soil-grout bond, thus increasing the pile capacity.

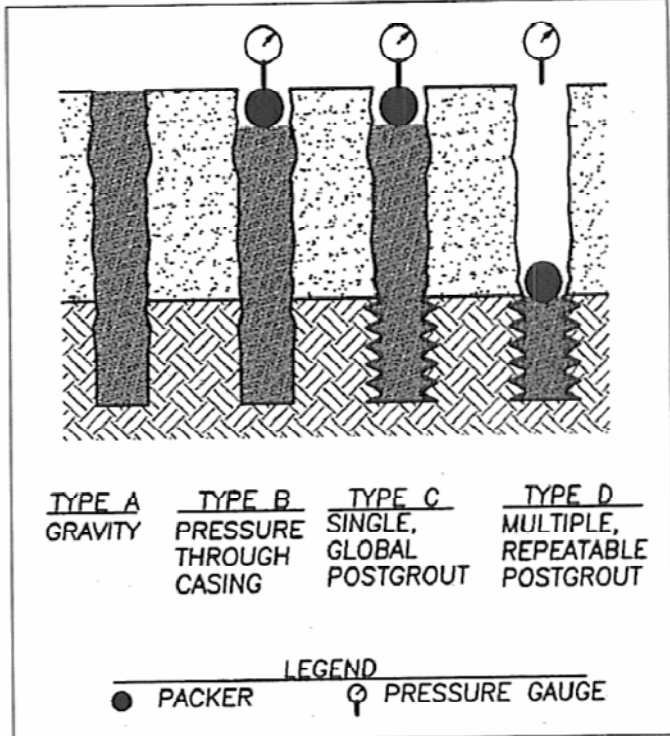


Fig. 3(b) — Classification based on construction (grouting) method (Bruce et al., 1995).

The structural strength of the existing slab was the first limitation to this project: underpinning of the machine could not be achieved through the use of Case 1 micropiles alone. Transferring the loads from the bearing pads of the machine with Case 1 piles would have required a structural element such as a grade beam or pile cap. Significant point loading of the existing slab would have resulted in a tensile or shear failure of the concrete. Therefore, the design load for each pile was limited to 5 to 10 tons (45 to 90 kN). Limited access to the frame also made the Case 1 pile concept impossible. Therefore, an array of Case 2 piles was selected to develop a stiffer soil mass beneath the frame. Fig. 4 and Fig. 5 show the layout of the piles.

The design of the piles considered both internal (structural) and external (geotechnical) capacities. The piles were designed to occupy a minimum 3 in. (76 mm) diameter hole drilled 10 ft (3 m) into the bedrock. A 1 in. (25 mm) diameter (No. 8) deformed steel reinforcing bar ( $f_y = 60$  ksi [414 MPa]) was to be installed as the pile was grouted and the drill casing withdrawn. Since permeation of the ground was not feasible, a low mobility mix was designed consisting of sand, cement, and fly ash to facilitate

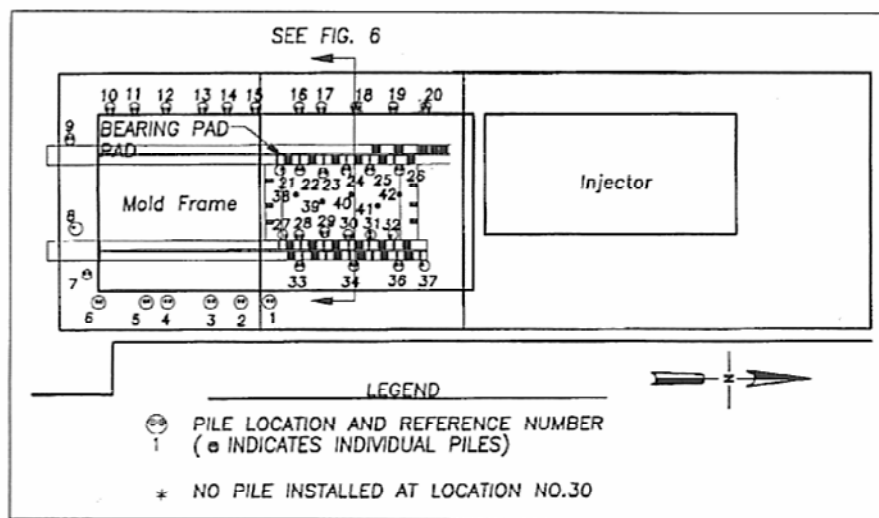


Fig. 4 — Micropile location plan.

cleanup within the facility. A bond strength of 200 psi (1.4 MPa) was used between the steel and concrete.

External capacity of the piles did not consider the contribution from the near surface soil and weathered rock. Although it is well understood that these materials contribute significantly to the capacity of a micropile (Bruce et al., 1993), bond capacity was conservatively analyzed by the grout-rock bond zone. The bond value was 15 psi (0.1 MPa) uniformly along the grout-rock contact, generating the required length of embedment of 10 ft (3 m). Due to the irregularities in the depth to rock and the consistency of the overlying

material, a standard length of 20 ft (6 m) below the ground surface was established.

The basic design considered micropiles installed around the perimeter of the equipment in groups: one vertically and one at about a 20-deg batter, angled under the machine. Additional low capacity Case 1 piles were located within the center of the mold area to accept direct load — both tensile and compressive — resulting from deformation of the slab. Case 1 piles were also used beneath support legs for ancillary equipment to limit additional stress influences in the northwest corner.

To further improve the stiffness of the slab-stone-soil system directly below the floor, it was decided to inject the area with a permeation grout to enhance its composite modulus. Initially, it

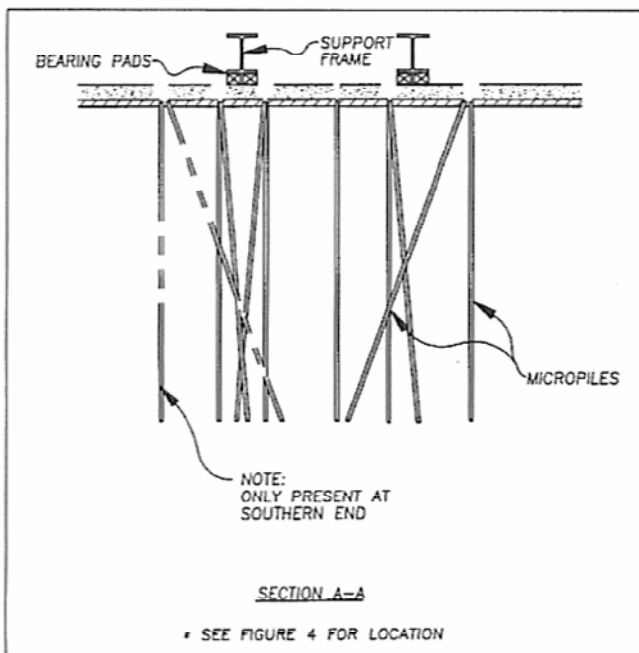


Fig. 5 — Micropile installation cross section.

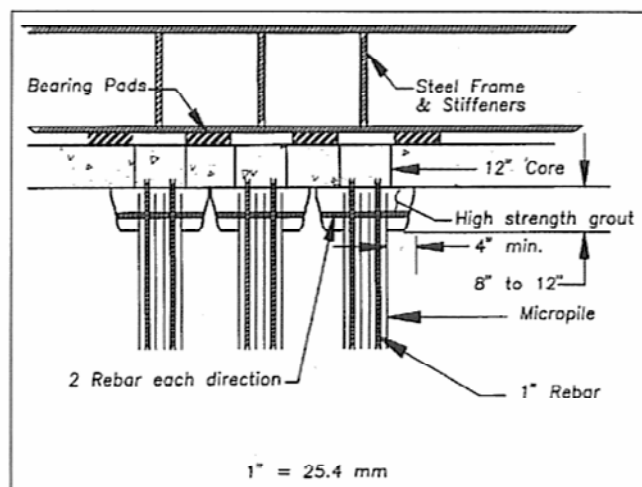


Fig. 6 — Micropile cap detail.

was unclear what type of grout would be effective in permeating the dense stone and soils. Several possibilities, including sodium silicate, microfine cement, and epoxy, were considered, each of which would fill voids within the material or along the contact of the layers and "cement" the material together to act as a single stiffer unit. Injection spacing and grout type were to be evaluated in the field based on the conditions encountered. This work was restricted to areas immediately adjacent to the bearing pad contacts, particularly in the transient load area.

## Construction

Construction began by covering the machine to limit contamination from the operation before coring 12 to 16 in. (300 to 400 mm) diameter holes through the floor slab to provide access for the drilling. Two micropile holes were drilled with a rotary track drill at each location to a depth of about 20 ft (6 m). A custom-made drill frame was used for holes inside the center of the machine. One large diameter cored hole was used for each pile group to reduce the amount of coring through the concrete slab. In addition, only half of the holes drilled adjacent to the bearing pads were allowed to be open at alternating locations to limit the risk of perforating the slab and causing it to crack, further reducing its strength.

At each of the 18 locations outside the extreme perimeter of the machine (No. 1 through 17 and No. 19; Fig. 4), one of the piles was vertical while the second pile was inclined at 20 deg from vertical towards the machine. At the remaining locations, the piles were



installed within 5 to 10 deg of vertical due to space limitations. Three piles were installed at Location No. 18 and No. 20 due to a particularly high concentrated load acting as both Case 1 for the ancillary equipment support leg and Case 2 for the machine frame foundation. Five additional 4 in. (100 mm) diameter holes were cored in the center of the machine area where the single Case 1 piles were installed.

Two inch (50 mm) inside diameter steel standpipe was installed in each hole to prevent caving following drilling. The reinforcement was then placed for the full depth of the hole. The holes were tremied full with at least 2 ft<sup>3</sup> (0.06 m<sup>3</sup>) of grout ( $f_c \geq 3000$  psi [20 MPa] at 28 days) under nominal pressure as the casing was slowly withdrawn. Grout was pumped into each hole with an Allentown Powercreter 20 pump through 2 in. (50 mm) inside diameter grout hoses while the pressures, volumes, and grout behavior (consistency and flow at the surface) were monitored to confirm the complete filling of each hole.

The steel standpipe was left in place at No. 18 and No. 20 to provide additional axial capacity. At these locations, the casing was lifted 5 to 10 ft (1.5 to 3 m) above the bottom of the hole to allow grout to flow around the outside of the pipe. The pipes were then redriven into the holes. Generally, the pipes were redriven with hand-held pneumatic hammers to within 3 ft (1 m) of the bottom of the hole.

The tops of the piles were cast into a cap to restore the floor slab (Fig. 6). Additional care was taken when the caps were constructed adjacent to the main equipment frame to limit the stress concentration and improve load transfer to the slab. At these locations and along the west side of the machine, the soil and stone in the area beneath the slab were dug out to a depth of 8 to 12 in. (200 to 300 mm) and 4 in. (100 mm) beyond the edge of the core holes. Subsequently, a high-early-strength cement mix was cast to fill the hole, creating a lip in the pile cap to transfer load from the floor to the piles. (Nominal horizontal reinforcing bars were included in the caps to provide some shear reinforcement.)

During coring of the floor slab, it was observed that water did not readily permeate into the subbase stone material beneath the slab. Therefore, the effectiveness of a permeation grout of any

type was questionable. However, following completion of the pile construction and repair of the slab, an epoxy resin grout was injected through 5/8 in. (16 mm) holes drilled into the slab with an electric hammer drill. Approximately 1/2 gal. (2 L) of grout was readily injected at each of the 23 locations around the frame. This material is believed to have filled some minor separations between the slab and the stone base, as well as some of the voids in the stone.

## Testing and performance

Following the underpinning, the machine was reconstructed and operations resumed. Measurements during releveling and loading indicated that deflections of up to 0.007 in. (0.2 mm) continued to occur. However, unlike the previous deflections, the movements were more uniform and did not seem to indicate that consolidation settlement or plunging of the northwest corner was occurring. Furthermore, no rocking of the slab was evident when the machine was in operation. Since the movements were rather uniform across the machine, they were not causing wear on the tiebars or misalignment of the injection head.

According to the client, within about a week of operation, measured movements were essentially zero, indicating that the equipment loading had been progressively transferred to the foundation system until a point of equilibrium had been reached.

## Final remarks

The installation of the micropiles and injection of the epoxy grout improved the overall stiffness of the system. Furthermore, the concentration of stresses that resulted in the plunging of the northwest corner of the equipment was eliminated. Thus, the remedial efforts are considered to have improved the foundation system, stabilized a weak structural system by stiffening the composite slab-soil system, and transferred a portion of the load to the underlying rock.

Accessibility limitations and construction time did not allow for load testing of the foundation elements; therefore, a conservative design was developed. Since the reassembly of the equipment was a significant endeavor, as much grout and as many underpinning elements as possible were

installed during the one and only breakdown period. The opportunity to install an engineered solution, test it, and add additional piles or grout if the testing failed was not available.

Additional evaluation of the floor slab and load testing are anticipated once the machine is removed as part of the final remediation and development of a foundation system in this area to allow continued assembly of new machinery without concern for foundation stability.

## References

- Bruce, D. A.; Bjorhovde, R.; and Kenny, J. R., 1993. "Fundamental Tests on the Performance of High-Capacity Pin Piles," *Proceedings, 18th Annual DFI Conference*, Pittsburgh, Oct. 18-20, 33 pp.
- Bruce, D. A.; DiMillio, A. F.; and Juran, I., 1995. "Introduction to Micropiles: An International Perspective," *Foundation Upgrading and Repair for Infrastructure Improvement*, ASCE Geotechnical Special Publication, No. 50, pp. 1-26.
- Warner, J., 1973. *Compaction Grouting*, ASCE JSMFD, August, pp. 589-601.

Selected for reader interest by the editors.

**Allen W. Cadden** is a senior associate with Schnabel Engineering Associates, and Manager of their West Chester, Pa., office. He is a geotechnical engineer focusing on the application of cement grouts for ground improvement and structural support. He received his Master's degree from Virginia Tech, Blacksburg, Va.

**Donald A. Bruce** received degrees in geology and civil engineering from Aberdeen University, Scotland. In 1996 he formed ECO Geosystems, the U.S. office of a Canadian consulting firm specializing in drilling, grouting, anchoring, micropiling, and soil mixing technologies. He also serves on numerous committees on dams, foundations, and grouting.

ACI member **Robert Traylor** is the division manager for the Baltimore, Md., based GeoStructural division of Structural Preservation Systems, Inc., the largest contractor of concrete repair in the United States. He received his Bachelor's degree in geology from Utah State University, and has 12 years experience in the structural repair and strengthening of commercial and industrial structures, including the use of micropiles.