Lizzi's Structural System Retrofit with Reticulated Internal Reinforcement Method

James A. Mason and Donald A. Bruce

The internal reinforcement method (IRM) was developed by Fernando Lizzi in the early 1950s. It is a complete structural system reinforcement method used to retrofit nonreinforced masonry and stone structures and any associated foundation systems. The general design principles of the foundation retrofit are similar to the principles used in the structure retrofit, for placement and engagement of reinforcement. The IRM technology integrates the complete structure, the foundation and bridge or building, into a continuously reinforced seismic-resistant system. The Three Arches Bridge is the only three-span bridge in Venice today. Built in the 17th century, this simple pedestrian bridge was constructed of nonreinforced clay brick masonry with spread footings bearing on the canal floor. Over time, the differential settlement caused localized distress to the overall structure. It was in such a state of disrepair that it was scheduled for demolition in the early 1960s. Lizzi was named to design and construct the historic preservation and structural retrofit of the complete structure. This was accomplished with reticulated internal structural stitching and reticulated micropiles. This structure is a case study of use of the IRM for static and seismic retrofit. The geometric simplicity of the bridge and the clarity of purpose and execution of the reticulated internal stitching and micropiles provide an example of technology that exemplifies the IRM method of integrated complete system static and seismic retrofit.

This paper introduces an internal strengthening method that can be used for the static or seismic upgrade of an existing unreinforced masonry structure and foundation system. All added reinforcement is contained within the structural components, that is, the retrofit is not external but is entirely internal. This technology integrates the complete structure so that a measure of stiffness along the height of the system, from the bottom of the foundation to the top of the building or bridge, has a smooth gradient. This strengthening system was invented by Fernando Lizzi of Naples, Italy, in the early 1950s (*1, 2*). Lizzi was a young engineer when he was presented with the problem of arresting the settlement-induced distress of numerous nonreinforced stone and nonreinforced masonry structures in Italy. He realized that to stabilize and stop the settlement-induced shear deformation of the building or bridge, the foundation system would have to be retrofitted. Lizzi used a top-down construction method (personal communication, F. Lizzi) that started from the superstructure and worked to the foundation. Furthermore, any work done on historic structures had to be visually imperceptible. To appreciate the genesis of the internal reinforcement method (IRM) technology, as developed by Lizzi, one first needs a perspective on the state of the art for underpinning and structural retrofitting around 1940.

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HISTORICAL BACKGROUND OF AMERICAN UNDERPINNING METHODS

In the United States, the rational development of underpinning for structures often is credited to the New York–based engineering firm Spencer, White, and Prentis. Their text, published in 1917 and updated in 1931 (*3*), set the course for many years. At that time, foundation engineers used rakers (shoring) and cross beams on cribbing with screw jacks to unload the foundation from the dead load of the structure, during the placement of underpinning; see Figure 1. Jacoby and Davis presented the needle-beam method in their 1917 work (*4*). They developed methods that would not disrupt the normal traffic and pedestrian flow into a building that was being retrofitted, which was a typical problem for many other methods of underpinning; see Figure 2. However, their underpinning techniques required excavation of the existing foundation system to construct the new system. The roots of this technique in underpinning work are found in the writings of Sooysmith (*5*) and Breuchaud (*6*), dating from the mid-1800s. This underpinning method required a substantial amount of excavation to allow new piles to be driven and to allow the construction of the associated pile cap, and so this method could potentially create three problems in addition to the original structural distress before any retrofit intervention: (*a*) the additional bending-induced shear stress created within the wall because of spanning the excavation opening in the soil directly adds to the distress of the structure; (*b*) unloading of soil during excavation creates a weakened zone in the soil mass that must be recompressed on unloading the temporary support system and subsequent reloading of the retrofitted foundation system; and (*c*) adjacent support soils could settle because of the dynamic vibrations of pile driving. Thus, the underpinning technique that uses the needle-beam system and associated piling driving techniques in fact causes additional distress to the overall structural system.

The distress caused to a building by the underpinning process can be seen in Figure 2. The structure's shell had to be punctured to allow the insertion of the needle beam. The adjacent rooms were abandoned during the complete construction process, and for businesses within the structure, the retrofit of the building meant relocation as a minimum action.

The required bearing of the needle beam within the plane of the wall for the vertical lifting of the building required infilling of the hole punched through the wall such that intimate contact between needle beam and the existing masonry was developed. The cumulative effect of the localized relaxation and subsequent compressive deformation in the region of the needle beam on reloading added additional shear deformation within the wall.

These examples of the development of the common methods for underpinning and superstructure retrofit show that current methods of

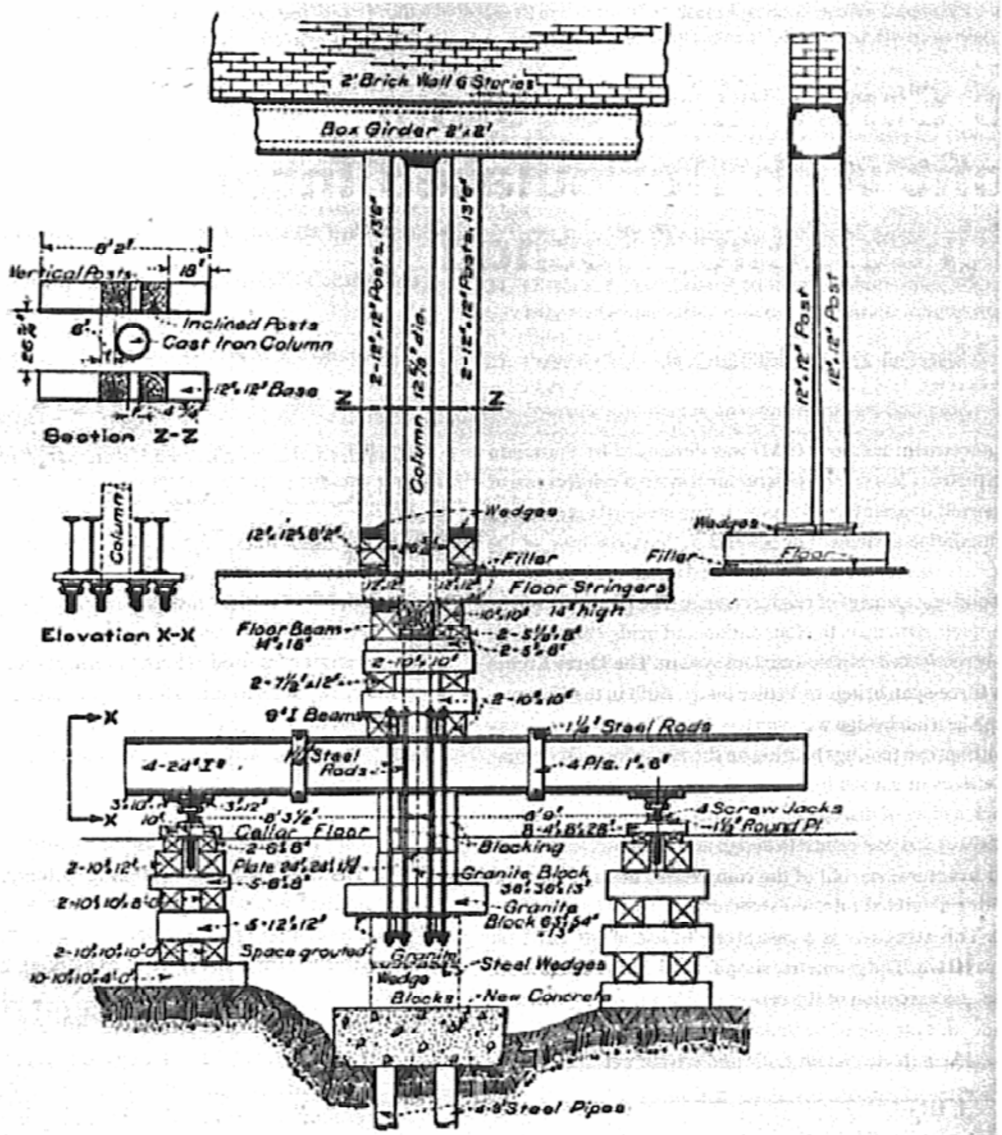


FIGURE 1 Early 1900s needle-beam underpinning detail (for "300-ton column on quicksand," Sargent Building, New York City) (*4*).

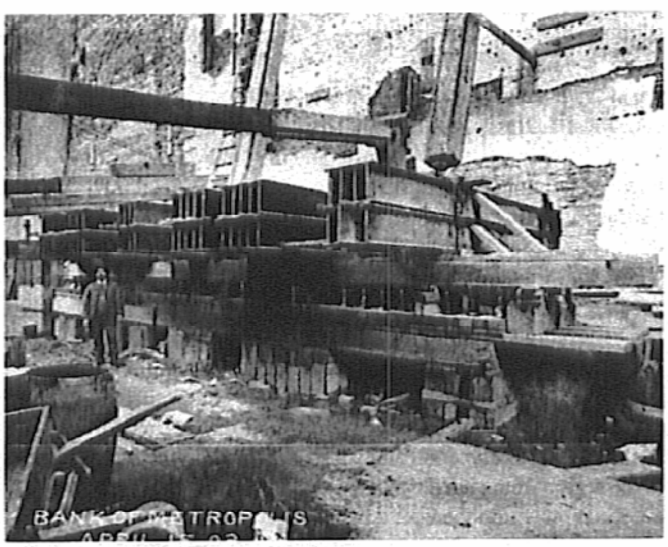


FIGURE 2 Underpinning the Metropolis Bank Building, San Francisco, 1902.

structural repair, directly descended from mid-1800s technology, could add distress to an existing unreinforced masonry building or bridge. Retrofit of structures of this type can be achieved without further significant construction-induced distress and—important for historic structures—without changing the original aesthetic of the structure (such as through installation of steel moment frames or attachment of fiber-reinforced composites to the external face of the structure). Just as important, many of the structures that Lizzi retrofitted in Italy have survived earthquakes of moderate to strong magnitude without structural damage (personal communication, F. Lizzi).

Lizzi and Development of Micropiles

The typical scheme for underpinning a wall of a structure was to install piles either some distance outside the face of the wall or in the plane of the wall. Both methods necessarily cause soil and structure relaxation, a negative condition in both cases. Also, piles installed outside the face of the wall induce further rotational distress to the structure because of the eccentric point loading between the pile and the wall. Lizzi realized that for an effective foundation and structure retrofit, both these conditions had to be overcome. The result is what we now know as reticulated micropiles and internal strengthening of the superstructure—in Lizzi's words, *pali radice* and *reticulo cementato* (*2*).

Design Concepts for Micropiles

Lizzi's original micropiles, the pali radice, are installed from the exterior ground or interior floor surface (*2*). There is no general need to excavate for temporary support, except for exploratory holes to confirm the size of existing foundation components. This type of micropile is constructed by drilling a hole to depth, inserting a centralized reinforcement bar, and gravity grouting the pile. For historic structures, the micropile usually is installed (i.e., drilled and grouted) through the existing wall. By using this method, the foundation system is engaged fully with the superstructure. The limitation that Lizzi imposed on the axial load of the pali radice is the crushing strength of the grout at the pile top. As the diameter of the cross section is increased, a proportional increase of side resistance per unit length of pile is developed with increased capacity. As the soil becomes stronger and stiffer, the pile length can be shortened to develop the same ultimate capacity. The service loading demand will dictate the pile length and spacing, based on this limit. For most of Lizzi's designs, it is apparent that he used a service level capacity of about one-half the ultimate capacity. This value is linked to the very linear elastic load shed performance of the piles.

The design process of a reticulated pali radice structure is discussed elsewhere (*7*). There are several ways to visualize the reticulated micropile network—for example, as an in situ soil reinforcement or as an in situ retaining wall. In any case, Lizzi emphasized and utilized in design one important component of the reticulated soil-structure system: the gravity resistance of the encased soil mass within the reticulated structure. The first part of the analysis for a reticulated micropile network is the model of an in situ retaining wall. Then, the analysis investigates the combined stresses because of gravity and active pressure that will load the retaining structure, as though there were no micropiles in the structure. Summaries of available methodologies are given by Clouterre (*8*) and Bruce and Juran (*1*).

Use of reticulated micropile networks for the underpinning of historic structures can be of two general geometric types, linear or closed, as discussed by Kulhawy and Mason (*9*). The linear configuration is an alternating installation of battered piles, which when viewed on end shows that the crossover (i.e., the node) is centered under the wall being underpinned. Kulhawy and Mason developed a language to describe the closed-form reticulated micropile network: a "quilted" soil arch develops from pile to pile, with "soil diamonds" integrating around the complete surface to form the quilt (*9*). This system can be thought of as a boundary surface and can be designed as such.

The design of individual micropiles should follow a rational design methodology, similar to that for drilled shaft foundations (*10*). Therefore, usually on-site full-scale load tests of individual micropiles are recommended, as well as any network configuration, if possible.

Critical to the design process is the evaluation of the operative horizontal stress after installation. Gravity grouting of the micropile is similar to that for a drilled shaft, but pressure grouting will increase this stress. (It must be emphasized that grouting of the internal reinforcement within the walls of a historic structure is a delicate process. This should be done at the lowest possible grout pressure, and gravity feed is the safest.) If the grout pressure effect is not included in the micropile resistance estimation, the capacity could be underestimated by 50 percent to 200 percent. Preliminary work by Kulhawy and Jeon illustrated this effect well (*11*).

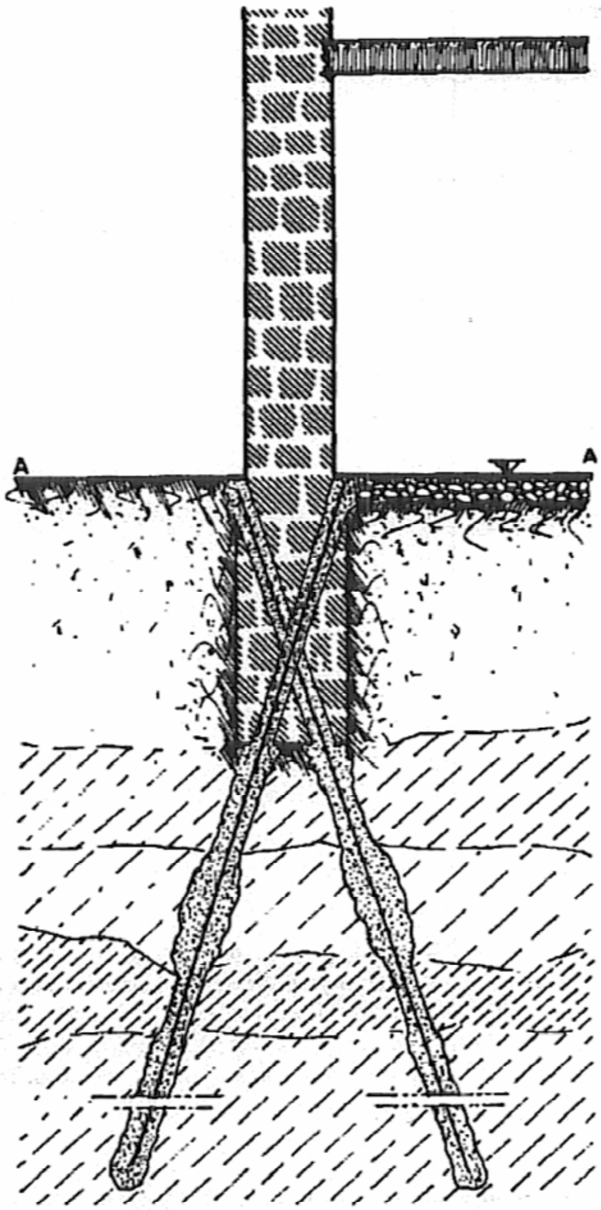
Kulhawy and Mason

The two general geometric types of reticulated micropile networks are those in a linear configuration and those in a closed configuration. The linear configuration is an alternating installation of battered piles, which when viewed on end show that the crossover, or the node, is centered under the wall being underpinned (Figure 3). The immediate benefit of this installation method is that the piles can be installed without excavation. The component that complements the node installation, as well as all other Lizzi methods, is the internal stitching that is installed in the wall (not shown in Figure 3). It is by integrated strengthening of the structure-foundation interface that the loading is carried smoothly through the structure and into the foundation system.

The design of the palo radice should follow a rational design method for drilled shaft foundations (*2, 10*). The need to include, for example, proper soil properties, stress states, overconsolidation effects, and geologic stratigraphy is basic to the process. However, the estimated capacity as predicted by integrating the side resistance can easily underpredict the actual capacity by 200 percent to 300 percent. Thus, on-site full-scale load tests of individual micropiles are always recommended, as are tests for any group configuration, if possible. The possible savings realized by load testing could easily substantiate any up-front costs associated with the testing.

At this point in the current understanding and modeling capabilities, the additional axial and lateral capacity due to soil arching within (for axial loading) and along the outside surface (for lateral loading) of a reticulated micropile network cannot be explicitly quantified. Therefore, designers who are aware of the arching mechanism think of it as a factor of safety to typical design methods. Lizzi showed that a net positive group reaction does exist within micropile networks (*2, 7*), whether they are installed vertically or in a reticulated pattern. The critical spacing for the vertical pile group is roughly at a 4-diameter spacing of piles. The efficiency is defined as the normalized group load divided by the individual pile capacity of the same length. There are two interesting facts to notice in these results: the pile group efficiency is greater than one, that is, a positive group effect, and as the piles get longer, the spacing at greatest efficiency gets larger. This suggests that positive arching occurs within the pile group with depth effects manifested in a greater clamping action to provide a greater arch thrust reaction.

The other basic component of a reticulated micropile network is the group action. Because of the arching action developed from pile to pile, the reticulated system becomes a closed surface encasing the interior soil mass; see Figure 4. This quilting action provides a positive group response to loading both for axial loading and for lateral loading (*9*). Again, Lizzi showed the positive group effects via model tests that he performed with vertically installed micropile networks in comparison with reticulated micropile networks with the same number of micropiles with the same length (*2*). These tests showed a multiplicative increase, that is, a positive group effect, of approximately 220 percent for the dry coarse sieved sand. Nothing was mentioned about whether the material was contractive or dilative, that is, the density state. And although structures Lizzi retrofitted with reticulated micropile networks have survived earthquake loading with no visible damage (personal communication, F. Lizzi), a quantifiable and codifiable methodology required for general geotechnical foundation design still does not exist.



(a)

(b)

FIGURE 3 Basic underpinning configuration with micropiles—the node: (*a*) vertical cross section; (*b*) horizontal cross section A-A (*2*).

INTERNAL RETICULATED STRUCTURAL STITCHING

The structural component to complement the reticulated foundation system is the reticulated structural stitching: reticulo cementato. Lizzi presented structural component retrofit reinforcement patterns for numerous structural components (2). The general patterns presented are pictorial, without giving the associated layout dimensioning or the necessary bar size. In general, the layout of the reticulated pattern is applied to both the front and the back of the structural element. The pattern is staggered enough from front to back to allow the effective tensile reinforcement to develop the tension demand as though the reinforcement were continuous. The bar size should be kept to a minimum so that an underreinforced section condition is developed. As the demand increases, the density of reticulated pattern groups should increase so that the installed area of steel necessary for moment and shear demands is met, as opposed to increasing the bar diameter. Obviously, common sense must play into the design in the context of overdrilling an area. Current research is developing design equations that can be used by the general structural and geotechnical engineering community. Because the mechanics of the IRM stitching can be thought of as a singly reinforced section, simple Whitney stress block–type equations are the expected methodology. Thus, it is expected that equations similar to those for reinforced concrete will be used for design. Figure 5 shows a typical reticulated reinforcement pattern for the retrofit of an arch. The crucial component that reticulation offers, over radial or straight bars, is that the reticulated reinforcement provides interlock of the various stone or masonry courses, a complete system integration. This is due to the three-dimensional engagement of adjacent courses with one another. This is similar to the interlocking that is developed in rock tunnel anchors with the various joint sets that intersect the tunnel walls. The joint sets for masonry are very regular for the external courses. Yet often in masonry structures, random-shaped material is used as fill between the walls. Reticulated reinforcement ties together the entire mass to perform as a continuous structure.

THREE ARCHES BRIDGE, VENICE

The Three Arches Bridge was constructed on the Rio di Cannaregio Canal in Venice, Italy, in 1688. The bridge was designed by Antonio Tirali (*12*) and was the winner of a design competition (*13*). It was constructed of

> . . . solid clay brick masonry in the same location of a previous timber bridge. As usual in Venice, the bridge had no parapets; two very stiff parapets were added during the first restoration in 1794; this fact modified completely the structural behavior of the monument, and the parapet acted as a stiff diaphragm. (*14*)

It is the only three-arch bridge in Venice today.

The general area of Venice is underlain by lightly overconsolidated Holocene fluvial deposits. The seismic risk for this area is minimal; various researchers have assigned values from "no risk" to "slight risk with an expected ground acceleration of approximately 4 to 8 percent *g* horizontal ground acceleration." (For seismic information on the Adriatic region, see http://www.dstn.it/ssn/index.html.)

The overall length of the bridge is approximately 40 m. The center span is approximately 15 m, and the two symmetric side spans are approximately 8 m long each. The original width was approximately 4 m. The voussoir thickness is approximately 700 mm. The two central piers are roughly 2.5 m wide and bear directly on the canal bottom (Figure 6). The original structure had a minimal amount of fill material placed above the pier to create the steps of the bridge. The structure has an arch thickness to radius of arch ratio

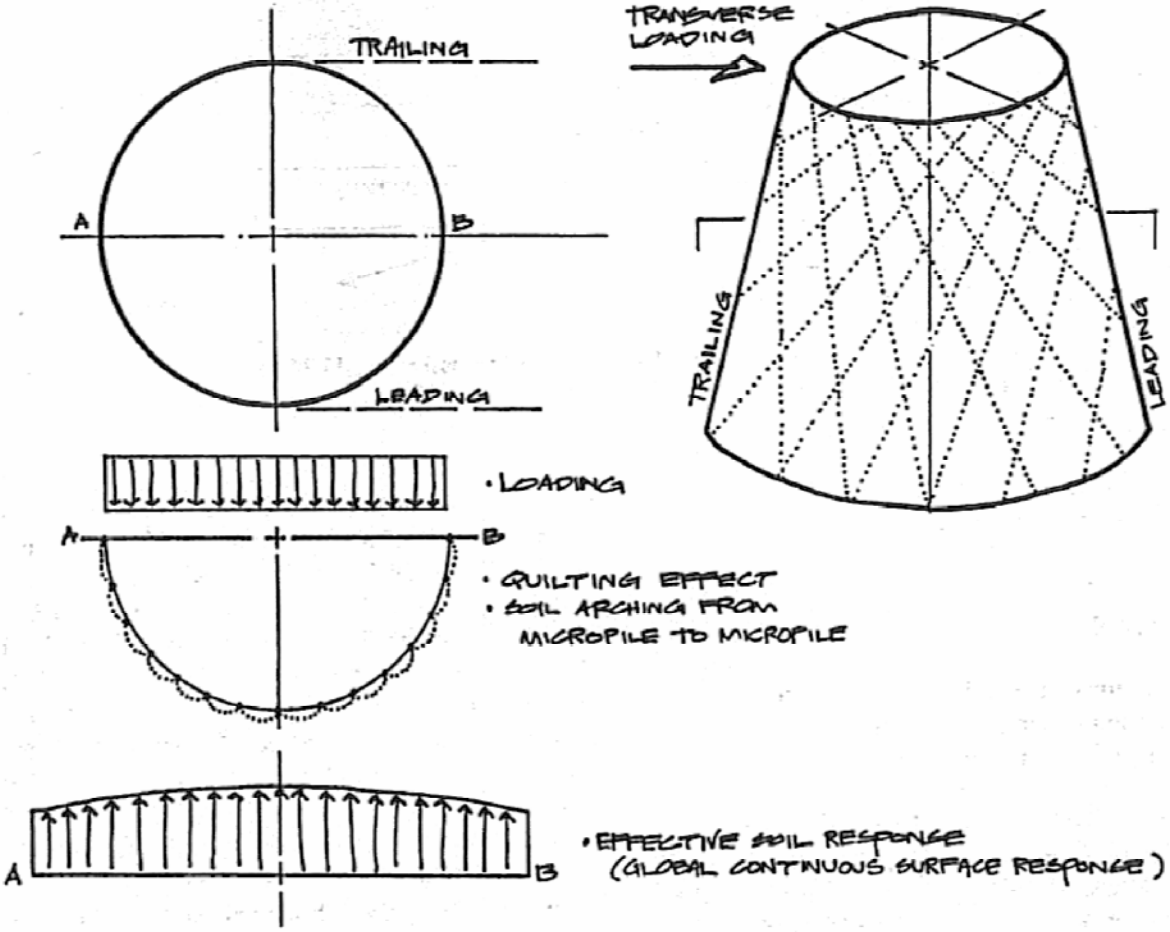


FIGURE 4 Global continuous surface responses of reticulated micropile network from soil arching quilting effect (*9*).

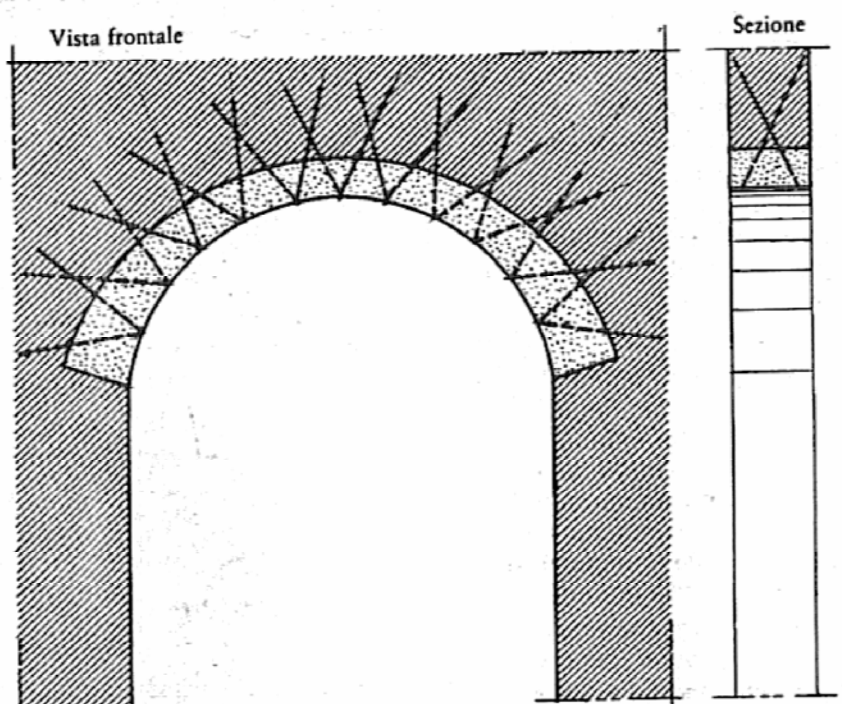


FIGURE 5 Typical reticulated reinforcement in arch (*2*).

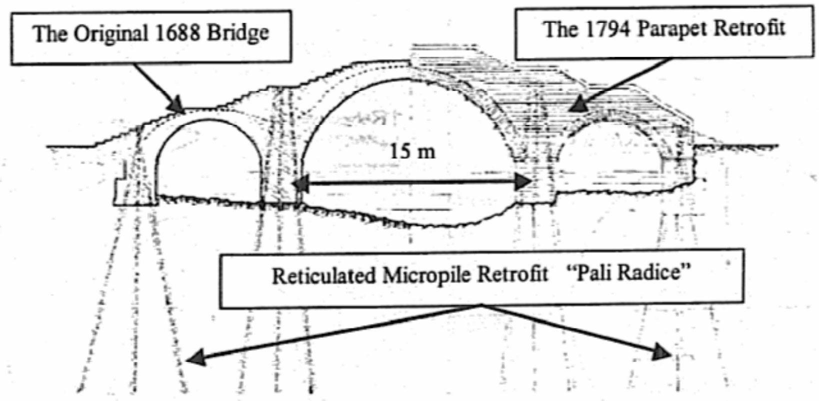


FIGURE 6 Composite view of original 1688 Three Arches Bridge, 1794 parapet retrofit, and reticulated micropile foundation retrofit in 1960.

equal to 0.1. This is in the optimum range for arch performance, per Heyman (*15*).

First Retrofit in 1794

In the 40 years after construction of the bridge, there had to have been enough differential support settlement to warrant the addition of the parapet walls. This amount of time for consolidation settlement of the Holocene deposits to manifest is very reasonable. The wall construction was three wythes wide with a cap beam along the top. The composite action of these relatively stiff parapet elements with the flexible arch was unbalanced. The parapets were not directly attached to the arch but were constructed on top of the edge of the arch. In time, the settlement was driven to the end-of-primary consolidation, and the structure became relatively stable.

Differential Settlement Distress

Because of a large increase in boat and gondola traffic on this canal in modern times, erosion of the support soils rapidly increased, leading to incompatible movement between the arches and the parapet walls. This differential movement caused tension cracking in the westerly end of the south parapet wall; see Figure 7. The amount of cracking caused sufficient concern for the officials of Venice that they considered demolishing the structure. Lizzi had been involved with other structures in Venice and with design plans for the restoration of the Venetian Lagoon itself (*16*). He was hired to design and construct the restoration of this famous bridge.

Modeling of As-Built 1960 Condition

The Venetian officials in charge of the restoration work required that a substantial effort be made to analyze the bridge. Lizzi knew that once the structure had been stitched with reticulated reinforcement, it would perform as a continuous, integrated structure, from the foundation up and including the superstructure, and the risk of seismic loading would be minimized (*17*). Lizzi's job was not modeling the as-built condition of the bridge. He modeled the retrofitted structure. But modeling of a nonreinforced masonry structure with a finite element package must accommodate the possibility of block slippage and movement, with appropriate kinematical models. One such approach is a modeling system known as the discontinuous deformation analysis method (*18*). This technology was developed to model tunneling in rock with all the discontinuities and joint sets that exist in that type of structure. The method appropriately allows the relative movement and slippage of the discrete components of the model. There is yet to be developed and confirmed a full three-dimensional package that can accommodate the closed-form structure of the reticulated foundation system with accurate three-phase soil modeling. Approximate models can be developed within two-dimensional (plane-strain) packages for undrained loading conditions (*16*). The coupling of the substructure with the superstructure is approximated for dynamic analysis with some finite element analysis (FEA) packages directly or with foundation springs, which approximate the foundation soil-structure interaction performance.

IRM RETROFIT SCHEME FOR THREE ARCHES BRIDGE

The overall goal for the IRM retrofit scheme was to create a structure that performed as an integrated continuous unit (Figure 8). Several components needed individual attention. First, the foundations were

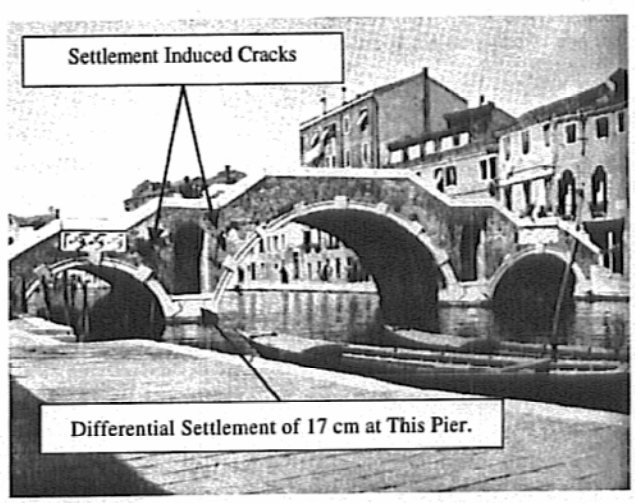


FIGURE 7 Differential settlement distress of Three Arches Bridge (*12*).

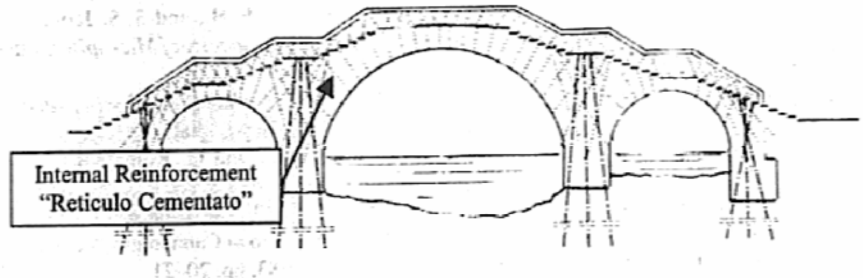


FIGURE 8 IRM retrofit scheme of Three Arches Bridge (*2*).

retrofit with reticulated micropiles. Then the parapets had to be stitched vertically together. The voussoirs were stitched together across the full width of the bridge, with the steel extending into fill material for tensile development, and a reinforced concrete countervault was constructed and attached under the existing arch. All the added reinforcement was reticulated. The countervault fulfilled two functions: general strengthening and environmental protection. Because of the boat traffic under the bridge, it was decided to eliminate possible direct contact with the arch material. Because there was relative movement between the bridge arches and the parapet walls, the parapet walls were tied to the arch through horizontal reticulated reinforcement. Once the IRM and foundation work had been done, a complete repointing of the structure was conducted.

A commercial finite element package was used to model the structure. The FEA model, seen in Figure 9, did not incorporate any interface elements, nor did it allow any independent movement of block subcomponents. The model forced compatibility of displacement at all nodes. Yet, for the completed structure, this was a correct assumption for the retrofitted structure. The foundation system was not included in the FEA. The loading for the FEA model was input as a different settlement of the westerly pier.

CONSTRUCTION ISSUES AND METHODS

Construction issues for foundation elements and steel reinforcement were some of the prime motivations for the development of micropiles and the reticulation technology. The need to be able to install a foundation element without excavation of the support soils gave rise to micropiles (pali radice). The need to install reinforcement into a nonreinforced wall section, with minimal distress during construction, led to reticulated reinforcement (reticulo cementato).

Installation of micropiles can be performed within the smallest of enclosures and with low overhead clearance (*1*). For the Three Arches Bridge, the drill rig was driven on top of the bridge, and the piles were installed from the top of the steps, down through the piers and into the substratum. Typical rotary drilling techniques were used. The piles were grouted for the full length, with structural connectivity developed in the region of the pier.

The parapet and arch reinforcement were installed from temporary scaffolding, allowing workers access to individual barrels of the bridge. Depending on the difficulty of drilling, that is, the hardness of the structural masonry, hand drills were used, or, for more difficult drilling, a reaction frame would have been installed and a ratchet system would have been attached to allow the driller leverage onto the drill. Lizzi used core drills extensively on all the projects for which he was responsible.

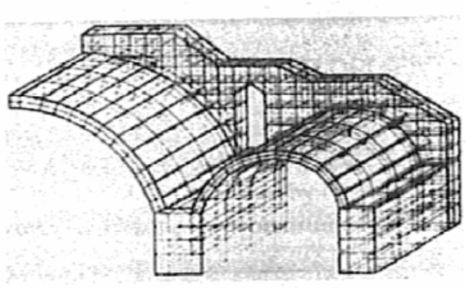


FIGURE 9 FEA model for Three Arches Bridge (*14*).

CONCLUSIONS

The development of reticulated foundation systems and reticulated structural reinforcement has added a valuable tool to the engineering community for the retrofit of unreinforced masonry and unreinforced stone structures. The mechanics of reticulation go beyond typical two-dimensional plane strain models, although the use of these two-dimensional equations provides insight into the possible structural performance. The design of reticulated foundation systems still relies on empirical methods, although research is in progress to develop rational design methods for both components. Most important, historic structures in Italy that have been retrofit with IRM have survived earthquakes by which nearby unreinforced masonry structures have suffered extreme distress.

The primary goal for this paper was to expose the structural and geotechnical engineering community to the IRM technology, because reticulation as a retrofitting method is just now becoming used in the United States. From these example projects, more opportunities will arise in which this technology can be used.

The Three Arches Bridge was successfully retrofit with IRM technology. The settlement-induced stresses were arrested at the foundation level via the reticulated micropiles. The structural continuity was developed via the internal stitching. The historic preservation and retrofit of the bridge was accomplished with minimal disturbance to the original aesthetic as designed by Tirali in 1688.

The development of the presented technology is credited directly to Fernando Lizzi. Because of his understanding of earthquakes and structural and geotechnical engineering, and because of his respect for the monuments of earlier master builders, a technology exists that provides a method for preservation of international treasures.

ACKNOWLEDGMENT

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