LIZZI'S PHILOSOPHY.
CONCEPTS TO PRACTICE.

James A. Mason, PE

Many seismic events in Italy have severely damaged historic structures in that country. These structures, buildings and bridges, are typically constructed of non-reinforced masonry or non-reinforced stone, possibly with inclusive steel or timber framing. Yet some of these structures appear to be surviving these extreme events with minimal damage. One retrofitting technology in particular has performed with exceptional resiliency. In the early 1950's, Dr. Fernando Lizzi, of Naples, Italy, invented a reinforcing scheme, which is placed within the physical boundaries of the original structural elements, with little or no disruption of the original design aesthetic. The internal structural reinforcement is combined with a unique foundation system that integrates the entire structure, foundation and superstructure, into a continuously reinforced system. Lizzi called the internal reinforcement "reticolo cementato", (cemented network), and the foundation system "Pali Radice", (root piles).

This paper presents the philosophical basis and methods of practice that Lizzi developed for the strengthening of historic structures, for both superstructure and foundation retrofit. This is accomplished by first developing the structural performance conditions that earthquake loading imposes on manmade unreinforced structures (buildings and bridges), a primary design condition for Lizzi. Then, details of component strengthening and integrated foundation retrofit are presented. Lastly, several cases studies are presented which exemplify the complete technique.

1. Introduction.

This paper presents a technology that was developed over 50 years ago. The internal reinforcement “reticolo cementato”, (cemented network), and the foundation system “pali radice”, (root piles), also known as the Internal Reinforcement Method (IRM) and Reticulated Root Piles (RRP), respectively, were invented by Dr. Fernando Lizzi, of Naples, Italy, to solve reinforcement and restoration problems for historic structure consolidation projects in Italy and the rest of Europe. The distress that is seen in many older structures is typically a combination of foundation movement (soil consolidation) in addition to weathering and deterioration of the original construction material. Lizzi understood that conventional underpinning methods would induce greater distress to a structure due to both the method of construction and the relatively large deformation required to mobilize the newly added foundation elements[1]. The combination of the structural retrofitting (IRM) and construction of a new foundation system (RRP) are the unique solution provided by Lizzi. The components of Lizzi’s technologies are installed in such a manner that they are not seen once the structural consolidation and restoration process is completed. The IRM and RRP components are constructed within the original structure elements, i.e., in the walls, columns, and foundations. They form an internal frame system which integrates the complete structure with foundation. The complete system provides a unique solution for historic structures requiring reinforcement for both static equilibrium and seismic retrofitting. Structures that Lizzi retrofit in this manner in Italy have survived scheduled demolition, due to material degradation and potential collapse, and earthquakes which would have been catastrophic otherwise.

2. Failure Modes for URM Structures.

Inherent weaknesses in unreinforced masonry (URM) and unreinforced stone (URS) structures are the discontinuous nature of the wall components and elements, and the lack of horizontal and vertical continuity of floor and roof diaphragms, from wall-to-wall and floor-to-floor (roof), respectively. The structure relies on the compressive capacity of the individual stone or masonry components with bedding mortar of various type and state, integrated into an agglutinated unit, for support of its gravity mass for vertical loading. The other basic structural components, provided by the same vertical load elements, are the shear, bending, and thrust resisting elements. A combined stress state could exist in the structure due to the self-weight of the structure and geometric placement of floors, openings within the walls, and arches. When a URM or URS structure is loaded by an earthquake, two other possible additional loading actions can be imposed on the structure due to seismic ground movements: 1) lateral loading, and 2) upward vertical loading. These two actions can manifest in-plane shear and bending, out-of-plane shear and bending, and torsional shear and bending. This section discusses typical failure modes of URM and URS structures and structural components due to earthquake ground movements.

Typical construction of walls for historic structures was of two distinct types: 1) Regular exterior block (outer wythe), either the same general size or general block shape of irregular size, with rubble in-fill between the exterior panels, or 2) Regular exterior block with partial or complete cross ties of either stone, or metal (lead or steel) bands extending through the rubble in-fill material. The in-fill rubble material was either partially or fully filled with cementitious dust or mortar. The wall width was empirically determined for assumed gravity loads. The composite action of the wall necessarily depends of the adhesion and interlock of the block elements and rubble in-fill to provide shear and bending resistance, minimal adhesion being provided by mortar, and interlock due to geometric interference of adjacent block. The weak or non-existent tensile capacity of the mortar bed gluing the components together forces the resistance of shear, bending, or torsion to wall component interlock. Figure 1 is a photo of a rubble in-fill wall in the Colosseum, Rome. The perimeter of regular block masonry surrounds the randomly placed in-fill material.

Figure 1. Rubble In-fill Wall. Colosseum. Rome.

Corners are one of the most important structural connections. If the corners fail, the structure begins to unzip. The two walls framing into the corner rely upon the connection to perform under a multitude of loading conditions. Corners are doubly loaded, the sum of adjacent wall displacement and rotation. The corner connection of wall panels relies on the inter-locking effect of adjacent elements from both walls meeting at the concurrent location. For static conditions, the connection capacity is the combination of the mortar adhesion and the gravity clamping force holding the corner header pieces. Under earthquake loading, the corners are vulnerable to horizontal rotation due to global torsional rotation combined with out-of-plane rotation of adjacent walls. The possible vertical displacement due to the seismic traveling waves adds to the inertial rotational effects just mentioned.

Large expanse of wall area becomes a weak component in a structural system during a seismic event. The inertial loading of the wall elements causes displacements normal to the surface, typically known as "out-of-plane bending". The connectivity of the wall to either a floor or roof diaphragm has direct effect on the magnitude of this displacement. The connection must resist both longitudinal shear and tension perpendicular to the wall surface. The ability of URM structures to resist out-of-plane bending displacements due to earthquake loading is minimal.

Another typical failure mode of URM and URS structures is the punching failure of floor and roof beams through the wall. The beams are typically perched on a bolster or in a formed hole in the wall. The seat length for beam or joist bearing is just a matter of several inches (25 to 100mm). The out-of-phase movement, due to seismic excitation, of the independent systems creates the possibility of the penetration of the timber members through the non-reinforced masonry, or the unseating of floor beams. The failure of the floor or roof diaphragm, by either punching or unseating, allows the unrestrained movement of the walls. This type of component failure is certain global zone failure.

Arches are vulnerable to lateral displacement which could dislodge the keystone, thus altering the inherent compressive stability of the arch. This could be both a static and a dynamic failure mode. The static case is due to lateral movement caused by subsidence (consolidation or erosion) of the foundation soils, or possibly, the deterioration of the arch abutment, within the wall, due to weathering. The dynamic earthquake displacements allow the arch to open with elements dropping vertically due to gravity, providing hinge points for bending failure of the arch. Figure 2 is the failure of unreinforced stone arches on the Stanford University campus due to the 1906 San Francisco Earthquake. The spandrels were constructed with regular exterior block with random infill.

Domes and vaulted barrel ceilings, i.e., thin shell structures, generally ornately adorned, are susceptible to seismic excitation. The slenderness of these elements provides little resistance to bending. The strength of this structure comes from the compressive capacity of the shell elements, the tensile resistance of the dome ring, or thrust resistance of the barrel abutment, respectively, necessary conditions for static stability. Thus the failure of this structure type is typically associated with displacements normal to the shell surface.

The failure of this element is typically catastrophic to the adjacent structural wall systems.

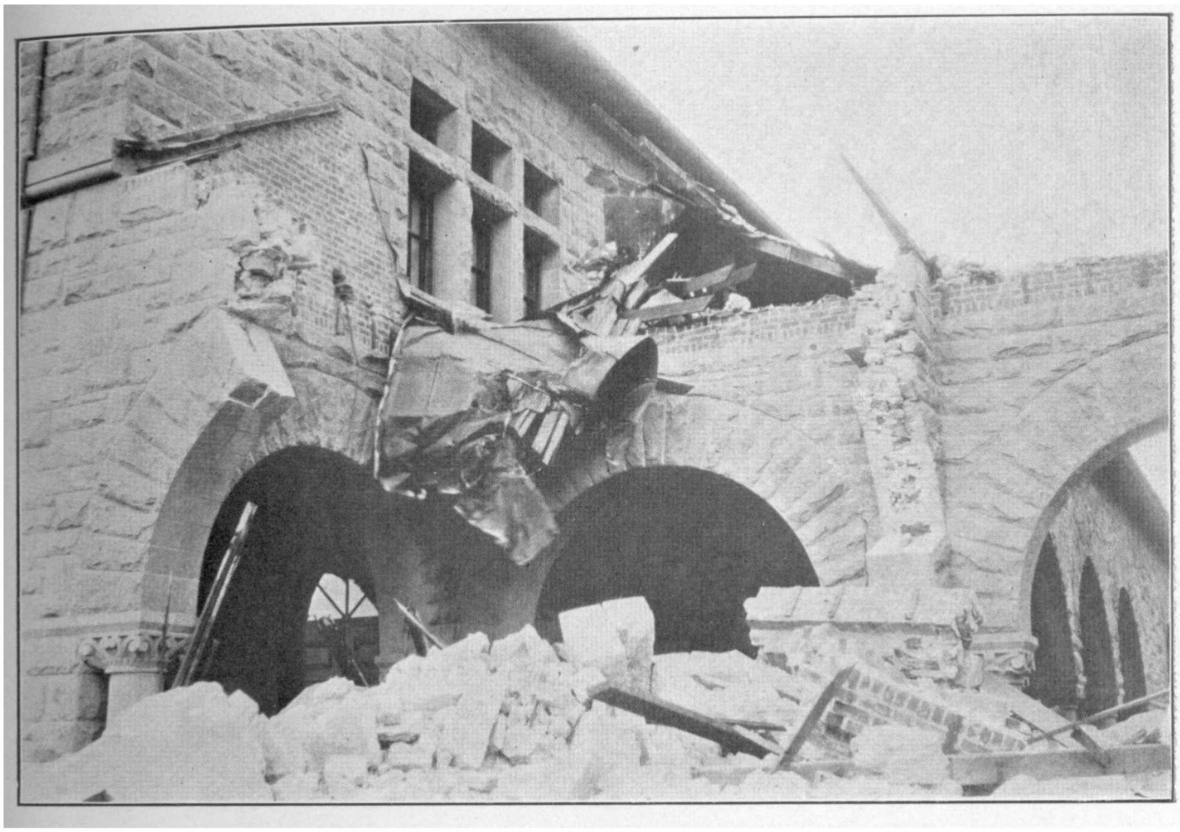

Figure 2. Stanford University. 1906 Earthquake Damage to URS Arch[2].

3. Lizzi's Philosophy for Retrofitting Historic Structures.

The philosophy of retrofitting historic structures for Lizzi was based in his respect for the original designers and constructors. His daily life was surrounded by the creations of master builders of Naples and Italy. It was in this environment that Lizzi developed a simple and succinct set of strategies that guided his actions.

- *Primum non necere. First, do no harm*.
- *Maintain the existing equilibrium*.
- *Reinforce both the soil and the existing structure*.
- *Strictly preserve the construction scheme and the original aesthetic designed by the original architect / engineer*.

4. Structures that Lizzi Retrofit.

Lizzi developed this technology in the late 1940's and obtained a patent for it in 1952. His career spanned over 40 years, with the majority of his design time being conducted for the Italian construction firm Fondedile. The number of structures that have been retrofit by Lizzi and this technology is substantial. The historical significance is realized by reviewing Table 1.

TABLE 1. STRUCTURES RETROFIT BY LIZZI[3].

1. The Church of St. Ferninando, Alvignano Caserta, Italy.
2. Milano. Portico dei Merchanti of the Palazzo della Ragione.
3. The Cathedral of Pienza.
4. Lamberhurst, England. Church of St. Mary (XIV Century)
5. England. Winchester Castle.
6. London. St. Stephen Church.

7. Venice. Ponte de Tre Archi.
8. Santa Vittoria in Matenano. The Town Tower.
9. Fano. Church of St. Marco.
10. The Ducal Palace of Urbino.
11. Venice. Danieli Hotel, (formally Dandolo Palace).
12. Venice. Manin Palace, seat of the Bank of Italy.
13. Rome. The Nero Aqueduct.
14. Bideford, England. 24 Arches Bridge (The Long Bridge), (XV Century).
15. Rome. Church of Santa Maria del Popolo.
16. Bari. The Old Town.
17. Paris. The St. Louis Bridge.
18. Dublin, Ireland. The Memorial Road Bridge.
19. Vertez Tours, France. The Bridge on the River Cher.
20. Rome. The Bonaparte Palace in the Piazza Venezia.
21. London. The Gospel Oak.
22. Derby, England. The St. Mary Bridge.
23. Bologna. The Palazzo della Mercanzia, (XVI Century).
24. Trapani. The Pepoli Museum.
25. Paris. The Notre Dame Cathedral.
26. Bath, England. The Downside Abbey.
27. Rome. St. Bartholomew’s Church on the Tiberine Island.
28. Parma. The University Palace.
29. Florence. Ponte a Cappiano. The Mediceo Bridge (XVI Century).
30. Palermo. The Chiaramonte Palace “Steri”.
31. Venice. The Hotel St. Marco Splendid.
32. Dumfries, Scotland. The Midsteeple.
33. Teruel, Spain. The Old Viaduct.
34. Lancashire, England. The St. Joseph Catholic Church.
35. Eure, France. The Monumental Church of Tourny (XV Century).
36. Gent, Belgium. The Het-toreken Building (XV Century).
37. Cambridge, England. The Ely Cathedral.

38. Ascoli Piceno, Italy. The Palace of Capitani del Popolo.
39. Alvignano, Italy. The San Ferdinando "Early-Christian" Church.
40. Amalfi, Italy. The Amalfi Cathedral.
41. Rome. The Church of St. Silvestro in Capite.
42. Paestum, Italy. The Temple of Ceres (500BC).
43. Naples, Italy. The Church of San Lorenzo Maggiore.
44. Naples, Italy. The A.Angiulli School (RRP only).
45. Rome. The Church of Sant'Andrea delle Fratte.
46. Florence, Italy. The Ponte Vecchio.
47. York, England. The Bootham Bar (XII Century)
48. Milan, Italy. RRP retaining walls for the subway.
49. Mendocino Pass, California. RRP retaining wall.
50. Paris. RRP retaining walls and structural underpinning.
51. Salerno, Italy. RRP underpinning for the New Italian Railway Tunnel.
52. Naples, Italy. RRP underpinning for highway tunnel.
53. Portovenere, Italy. The old Church of St. Peter (XII Century).
54. Agrigento, Italy. The Agrigento Cathedral (XI Century).
55. Nicosia, Italy. The Nicosia Cathedral (XIV Centruy).
56. Spilimbergo, Italy. The Spilimbergo Cathedral (XII Century).
57. Naples, Italy. Retrofit of a new building with IRM and RRP.
58. Burano, Italy. The Burano Tower (XVI Centruy).
59. Mosul, Irag. The Al-Hadba Minaret (XII Century)
60. Tokyo, Japan. The Panorama Tower.

5. Typical Structural Details [3].

The overall objective of the IRM technology is to integrate the existing unreinforced structure into an appropriately detailed continuously reinforced masonry or stone structure. The technique effectively creates an internal frame system with the retrofit components within the fabric of the original structure. The IRM is a drill and bond technique of installing internal steel reinforcement into an existing structure. The bars are installed in a three dimensional diagonal cross-stitching pattern. For most of the IRM component patterns, there is a front face installation with an associated back face installation. Adjacent rows are mirror reflected in location and direction with appropriate vertical stagger to avoid interference. By appropriate over-lap and spacing of individual bars, a doubly-reinforced section is created by the reinforcement, with the tension demand being distributed to the individual reinforcement bars. Small diameter reinforcing steel bars (1/2 in. (12mm)) work better than larger diameter bars at integration of the wall components and are the suggested tension element by Lizzi. Also, an under-reinforced section must be created to maintain the ductility necessary for earthquake loading. An engineered drill and bond installation method is utilized for location and placement of the reinforcement. Grout strength should be matched to the wall compressive strength. At present, generic design guidelines for IRM do not exist. A clear and methodical understanding of the 3-dimensional interaction of the component bars with the existing encasing composite wall is required for any engineered solution. One way of visualizing this technology is by the “strut-and-tie” method. Otherwise, the traditional moment couple method, compression block balanced by tensile resistance, ("Whitney Stress Block") is also appropriate for description of the mechanics. Shear reinforcement would be added if the shear demand exceeded the capacity of the grout bed interface at critical locations, as per normal methods that are utilized in reinforced masonry design. The use of the IRM technology can be applied to all components of a historic structure. At the completion of the consolidation process, the retrofitted and restored structure retains the original design aesthetic without a secondary external frame system. This section describes the various IRM details with plan and profile generic images for several of the specific components.

The retrofit methodology is a "top-down" process. A structural retrofit would begin at the parapet walls at the roof level and progressively move down the structure to the foundation retrofit.

Columns.

Natural stone columns inherently have inclusive seams and fractures. The original builders would have been selective in the choice of material to minimize the deleterious effects of the natural imperfections. Yet, time (weathering) and alteration of the original loading conditions could cause deformations which would impact the basic stability of these critical structural elements.

The retrofit of a column requires the systematic circumferential placement of reinforcement to provide both the confinement of core material plus tensile reinforcement for lateral loading (bending) induced stresses, see Figure 3. The spacing of the bars would be similar to the pitch of spiral reinforcement for reinforced concrete columns with vertical inclination adjusted for the specific column diameter. The same methodology can be applied to columns constructed from either stone block or masonry brick. The mortar beds can be thought of as joint sets in natural stone.

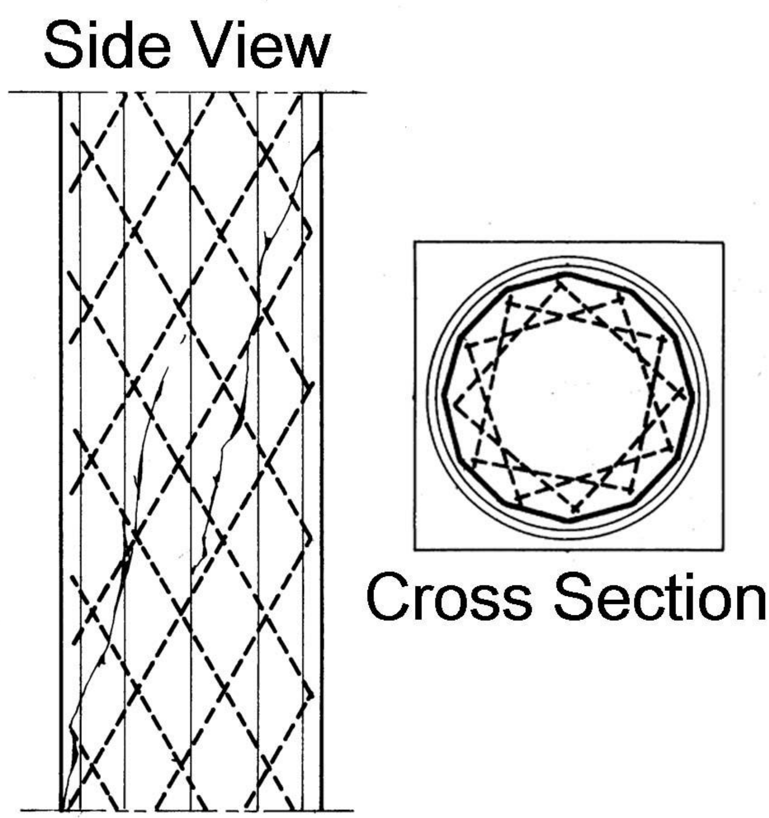

Figure 3. Column Retrofit with IRM [2].

Edge Beams.

An edge beam is constructed to provide both lateral and vertical bending resistance. It can be placed at the top of a colonnade or at the junction of a floor to a wall, see Figure 4. A continuous edge beam will provide a “girdling” effect at the specific reinforcement level providing restraint to displacements perpendicular to the wall, and reducing the vertical length of unreinforced open wall. The depth of the beam for vertical loading is designed to resist imposed gravity and possible vertical seismic forces. The depth of the beam for horizontal loading is limited by the depth of the existing wall section. The distribution of lateral out-of-plane loading from the floor or roof diaphragm can be resisted by the integrated equivalent plate action of the IRM reinforced wall. The edge beam is also designed for longitudinal shear forces due to seismically induced inertial horizontal floor diaphragm or roof forces.

Corner Reinforcement.

Corners are critical for continuity of wall sections. If a corner fails, stability of all adjacent elements is questionable. Thus, the retrofit of corners is paramount to basic stability. The placement of reinforcement ties the individual elements together from one wall to the other, vertically and horizontally, see Figure 5. Lizzi’s corner reinforcement also provides resistance to torsional rotation and resultant shear that occurs in tower structures. The length of the bar should extend into the adjacent IRM reinforced wall panels.

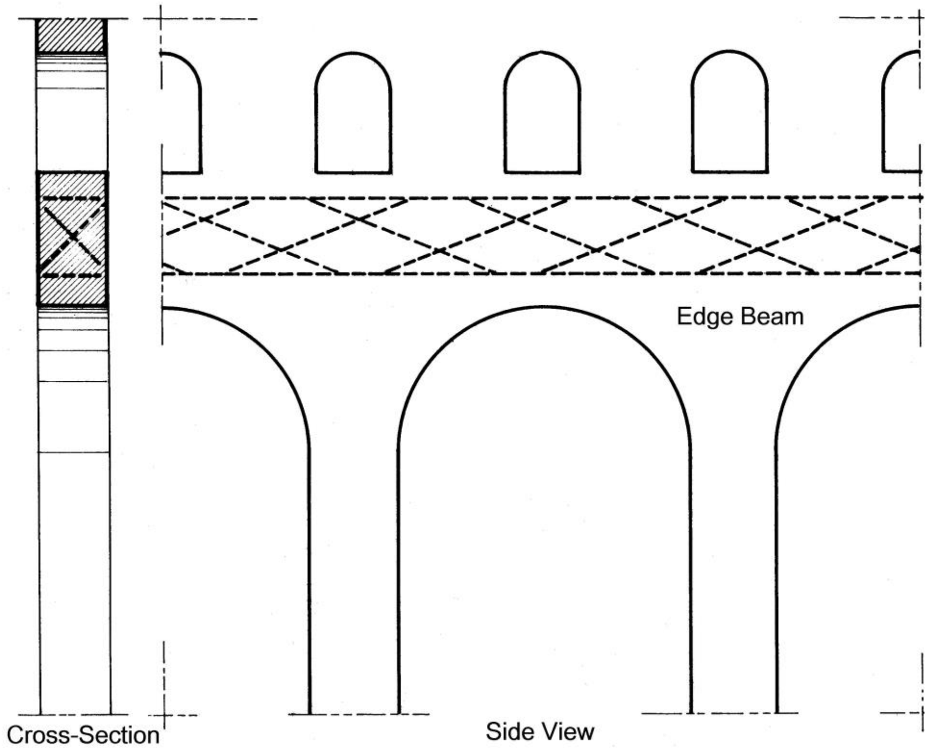

Figure 4. Edge Beam – Colannade IRM Retrofit [2].

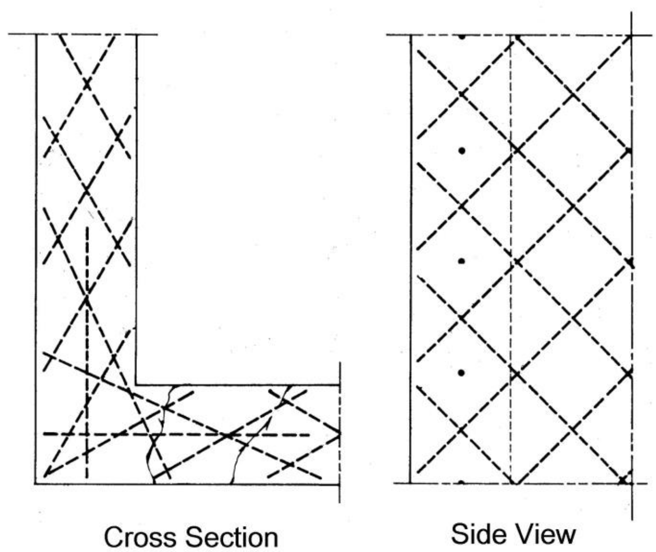

Figure 5. Corner IRM Retrofit [2].

Arch Reinforcement.

The preferred mechanics of an arch is to have all individual elements in compression. When settlement induced displacements shift the compression thrust line[4] out of the main arch mechanism, tension cracks and openings are created in the structure. This establishes a multi-beam strutted arch which is of minimal stability. The reinforcement scheme of the IRM is to tie together the arch elements with adjacent elements to create a reinforced arch beam with an effective depth of the length of the reinforcement. The arch must also be tied into the abutment or spandrel to create structural continuity.

Bolsters.

The design of reinforcement of a bolster must consider the shear demand at the wall interface and the tensile demand at the top plate due to rotation about the bottom edge. Diagonal bars are placed parallel to the outer surface as additional compression struts. All bars crossing the shear plane, with appropriate development length embedment in the adjacent wall, provide shear friction clamping at the wall surface and should be designed as such.

Floor Diaphragm to Wall Connection.

The design of the floor diaphragm to wall connection must provide the necessary connectivity of floor beams for vertical forces, and in both horizontal planar directions. The in-plane loading would be for normal (perpendicular) and shear (parallel) directions to the wall. This connection would integrate into the edge beam reinforcement. The possibility of punching of floor beams must be addressed in this design.

Open field, Out-of-Plane Reinforcement.

The general wall-open field reinforcement is designed to resist out-of-plane bending. This reinforcement pattern prevents large areas, such as end gables or long walls from failing due to inertial displacements perpendicular to the wall surface. The open field reinforcement must tie into other global reinforced zones to provide a connected and competent load path.

Domes and Barrel Ceilings.

Domes and barrel ceilings are both shell structures that are weak under displacements perpendicular to their surface. The placement of reinforcement into this type of structure demands extreme care during the installation. Due to the thin skin of the dome or barrel ceiling, the embedded reinforcement might have to be tied into an additional cast-in-place concrete shell placed above the original element, one of the only times that Lizzi used additional material for strengthening. Inclusive to this retrofit, it is necessary to provide a competent dome ring or ceiling abutment for the thrust reaction. Also, the added reinforcing steel is skewed in two directions to provide the interlock of the typical “reticulo cementato”, i.e., none of the added reinforcing steel is normal to the structure surface.

Parapets.

The design for the retrofit of parapets is based on IRM cantilever columns with tributary wall space in-between the columns. The spacing of the IRM columns is based on demand relative to capacity. The open wall space is retrofit as open field wall areas.

6. Typical Foundation Details.

Lizzi realized that most distress to historic structures was due to the settlement of supporting soils. The other motivating factor for development of pali radice was providing tensile reinforcement in the foundation to provide overturning restraint and stability to the main structure. The typical structure foundation type were spread footings as the main foundation element. Venice is a special case with timber pile foundations as the typical structural supporting system. As consolidation of the subsurface soils progress in time, uneven vertical displacement could manifest as structural displacements and associated cracking. From this knowledge, Dr. Lizzi developed micropiles, "pali radice" (root piles). These are small diameter drilled piles with cast-in-place concrete as the main bearing structural element. Typically a small diameter steel bar was placed in the center of the hole, full length, prior to casting. This bar is utilized to tie the pile into the base of the column or into the pilecap.

Lizzi's foundation retrofit is of four basic types [5] ,[6]: 1) vertical and sub-vertical, 2) linear node configuration, 3) linear reticulated wall configuration and 4) closed form reticulated configuration. A brief description of each system follows in the subsequent paragraphs.

The vertical "palo radice" (root pile) are used for column support for columns with competent base blocks. If the base of the column was constructed of individual pieces, then the micropile was drilled through the base in a reticulated subvertical pattern. This cross-stitching ties the foundation mass together into a unified block. The inclination and location of installation of the micropiles will be column dependent, as the material and construction of each column is unique. The center of reaction of the pile group must be coincident with the center of gravity of the column. This is to prevent any rotation, i.e., development of moment, of the column due to the eccentricity between the center of gravity of the pile group with respect to the base of the column. Figure 6 presents micropiles installed in a subvertical pattern to support an interior column of a building.

The linear foundation system is constructed of micropiles being installed perpendicular to and along the length of the wall inclined such that they go under the wall. Adjacent piles alternate inside to outside installation and alternating batter. The spacing of the piles should be close enough to provide positive soil arching under lateral loading. Looking at a cross-section of the wall with the installation method just described, the piles would form an “X” pattern, a node configuration. This orientation of micropile provides resistance to two actions: axial and lateral loading. Care must be given to the angle of batter of the pile. If battered, i.e., inclined, too far with respect to vertical then the axial loading would shift from axial loading to bending. This would be an ineffective use of the structural section of the micropile, as the pile is intended to be loaded axial with minimal bending. Figure 7 presents a nodal configuration.

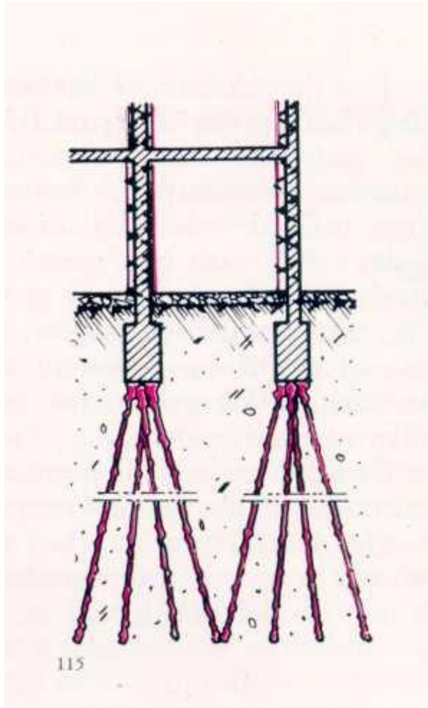

Figure 6. Columns Support on Subvertical Micropiles.

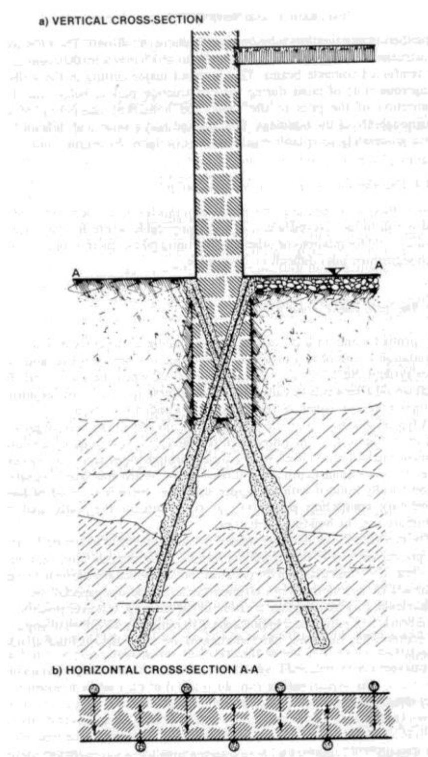

Figure 7. Linear Node Configuration of Palo Radice.

The third type of micropile group, developed by Lizzi, was the linear reticulated wall configuration. This type of configuration is utilized for urban retaining walls and slope stabilization, where a front and back (inner and outer) wall component is integrated into a retaining structure, i.e., a pile-soil system. The soil arching from adjacent micropiles (palo radice) in each wall forms a "soil quilt" which develops a resisting soil-surface perpendicular to the pile layout and anticipated soil movement. The main pile group, which is the front and back wall, and associated soil arching interaction, restrains the soil mass by physically blocking the movement and thus creating a gravity mass that becomes a major component of the in-situ retaining wall system. Also, the back wall restrains the active soil wedge that develops on the backside of the system. The active wedge is the primary loading on the RRP retaining wall. The combined action of pile and restrained gravity soil mass integrate into a composite that generates system stability greater than the action of the pile system alone. Figure 8 presents a generic layout pattern of the linear reticulated wall. There is an outer and an inner row of micropiles with the same inclination angle α, and with mirror symmetric pile batter angle β. The row spacing is designated by "s" times "d", where "s" is some fraction and "d" is the micropile diameter. The pile spacing is designated by "n" times "d", where "n" is some fraction and "d" is the micropile diameter. The row and pile spacing are functions of the pile diameter and soil type. This wall configuration is combined as a leading (front) and a trailing (back) pile group wall system to create the retaining wall. Figure 9 presents the reticulated micropile retaining wall in a cross-section view. The leading pile group is shown as A-A' and the trailing pile group as B-B'. The purpose of the piles in the center of the group are twofold: first, to integrate the interior of the soil mass contained within the leading and trailing walls, secondly, to provide additional vertical load capacity as required for the specific conditions. Excavation occurs in front of the leading wall. A rigid reinforced concrete cap is placed on top of the piles to restrain pile head movement. The pile to pilecap connection must be designed to accommodate any possible future lateral loading on to the cap, such as from up slope soil movement. The structural connection is designed for shear at the interface and sufficient embedment depth to develop a fixed head condition. Analysis of lateral loading due to the restrained soil mass must address bending demand onto the micropile. The micropile section is designed to have sufficient capacity to resist the factored bending load.

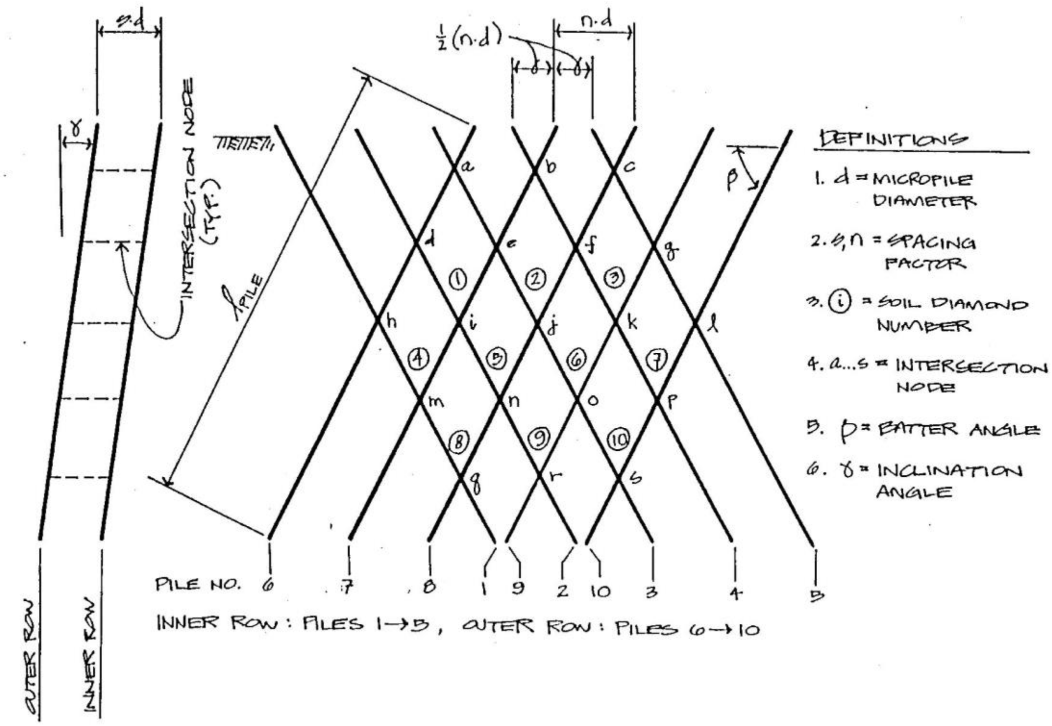

Figure 8. Quilted Wall. Front and Side View[7].

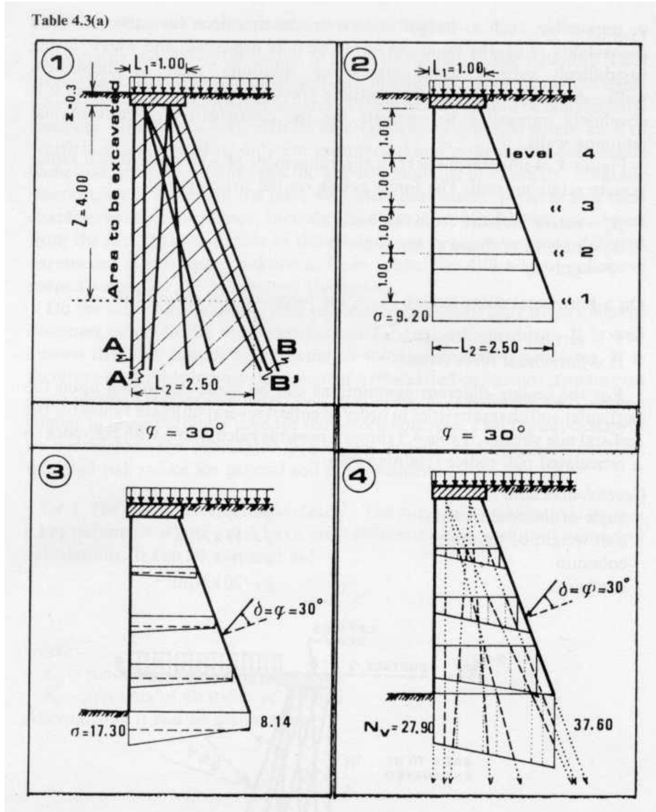

Figure 9. Reticulated Micropile Retaining Wall[8].

Figures 10 to 12 were generated with AutoCAD to show the three-dimensional nature of a RRP retaining wall. Figure 10 is shown as a cross-section of the wall system, in a similar fashion to Figure 9. Figure 11 is an image of the wall rotated approximately 30°. The typical diamond pattern of the RRP geometry is shown in this figure. Figure 12 is a front view of the leading (front) wall of the RRP retaining wall. The similarity to Figure 8 is seen in this figure.

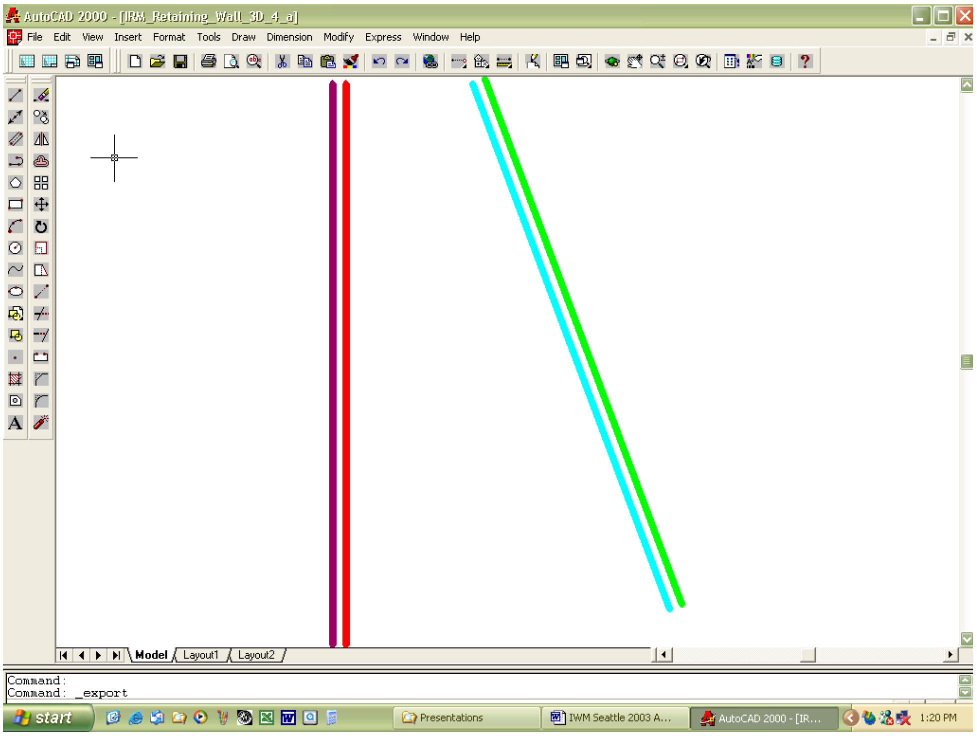

Figure 10. Reticulated Micropile Retaining Wall.

Cross-sectional View

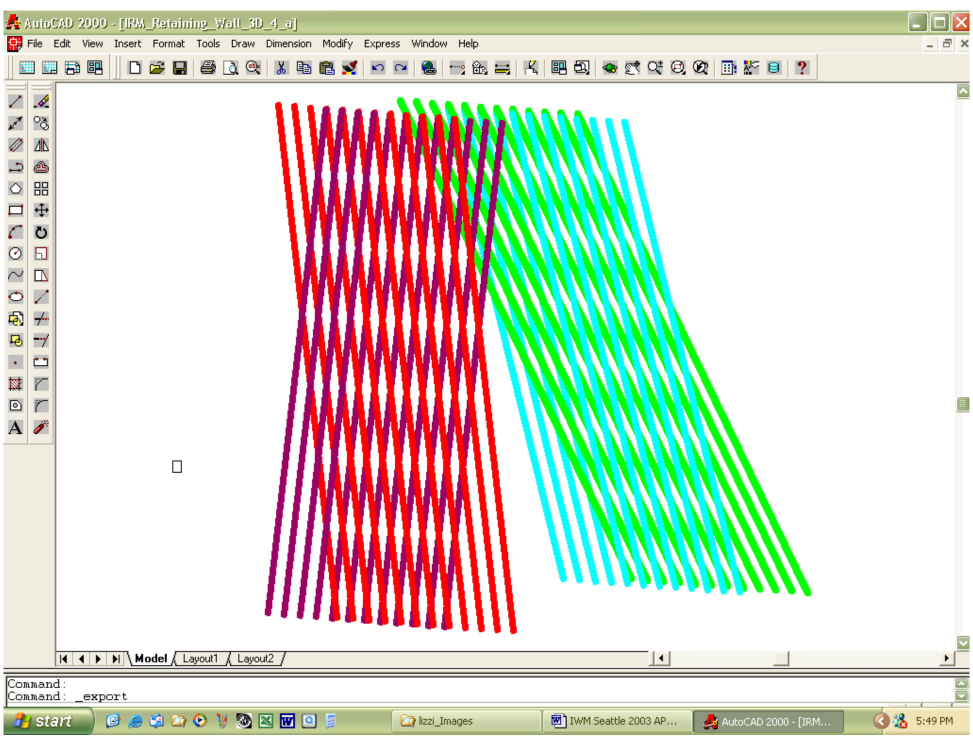

Figure 11. Reticulated Micropile Retaining Wall.

Rotation approximately 30°.

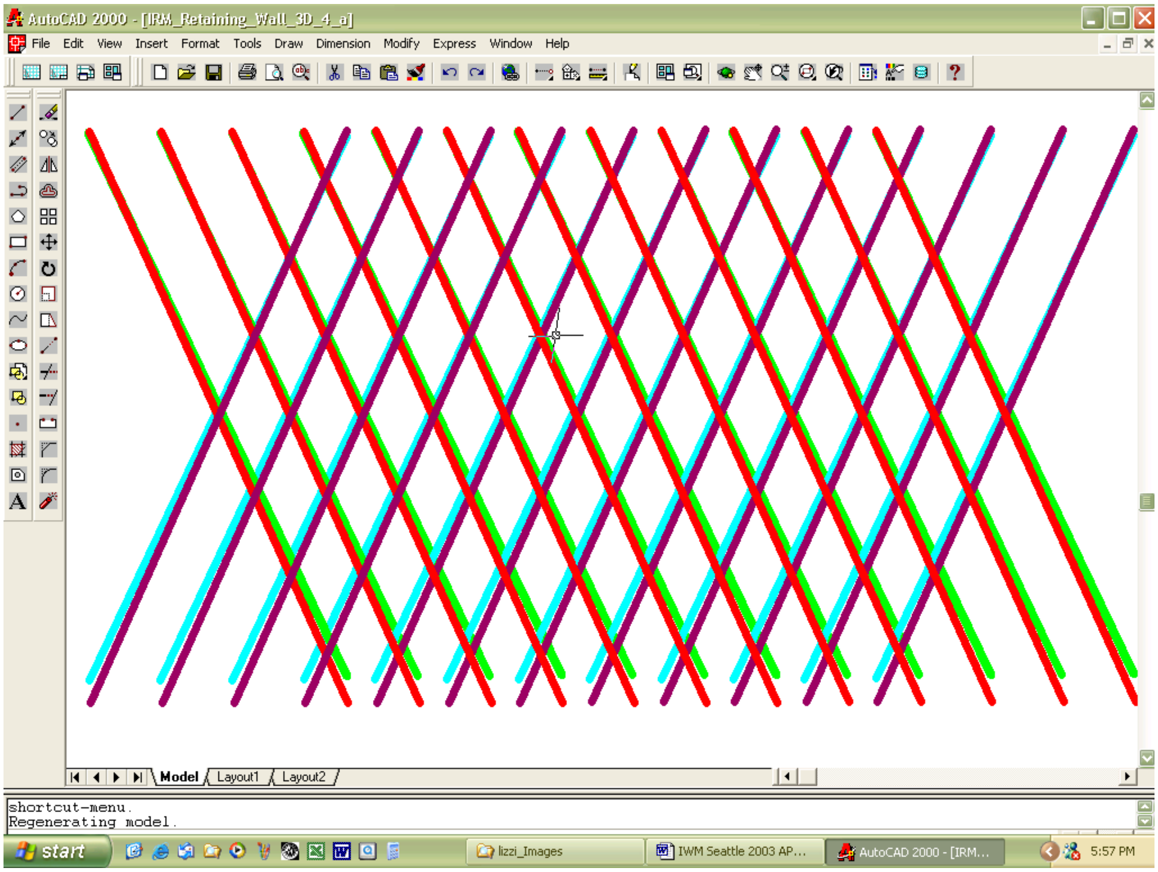

Figure 12. Reticulated Micropile Retaining Wall.

Front View of Leading Wall.

The fourth type of general configuration of a RRP foundation system is the closed form pile group. This configuration encases a soil mass, in the same “quilting” fashion as the planar reticulated wall. The sloping batter adds to the vertical and lateral resistance of the pile group. See Figure 13.

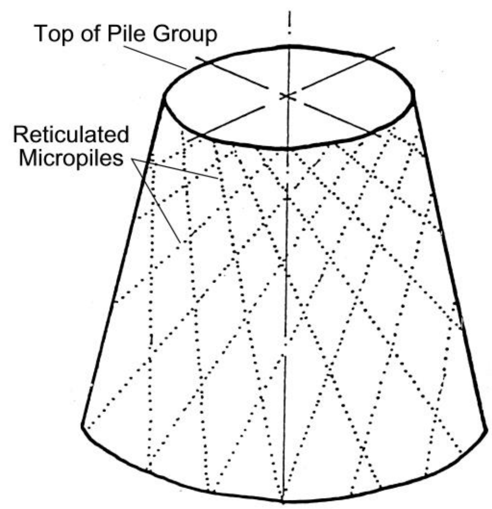

Figure 13. Schematic Drawing of a Reticulated Micropile Group[9].

One of the primary functions of this configuration is to constrain the lateral bulging of adjacent soils under a spread footing, thus limiting settlement displacement. As a vertical load is applied to a spread footing, settlement occurs due to two mechanisms; settlement due to compression, and settlement due to lateral bulging. This issue was addressed by Terzaghi[10] in his early writings, and is shown in Figure 14. Lizzi had attended lectures by Terzaghi in Europe, and incorporated modern soil mechanics, as presented by Terzaghi, in the development of the RRP foundation systems[11] and subsequent design projects.

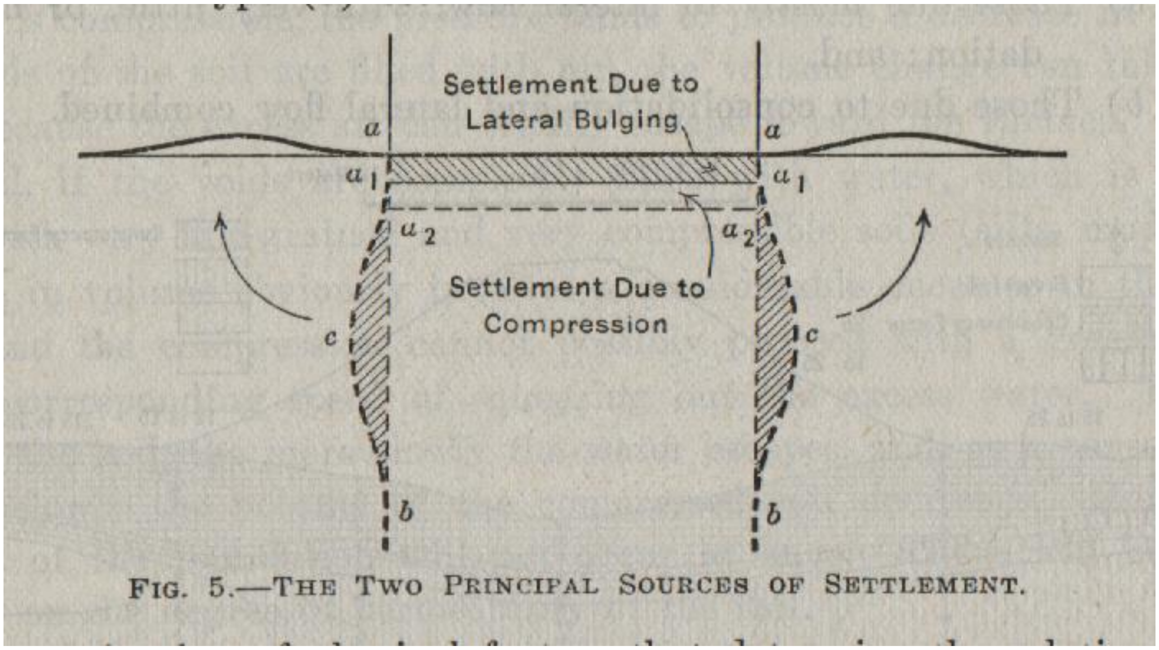

Figure 14. Settlement under a spread footing. Terzaghi (ASCE, 1929)

7. Integrated Structural Performance.

This system of structural retrofit with IRM and RRP was designed by Lizzi, from its genesis, as an integrated system. The process of tying together the superstructure with the foundation for a completely reinforced integrated system was the goal of this technique. The aggregation of component stiffnesses into system stiffness along the complete vertical length of the structure, from the bottom of foundation to the top of the building or bridge, provides a smooth structural transition along the complete length. For typical structures not designed in this technique, hard spots occur at specific locations; for example, the foundation to pilecap transition, at floor to floor transitions, or at the roof level. These stiffness jumps create locations (stress risers) that manifest in a substantial difference of deflection under dynamic loading. The displacement performance of a structure retrofit with both IRM and RRP, because of the smooth stiffness gradient bottom to top, is even and gradual in distribution. This creates a structure that will spread out dynamic loads efficiently along the complete length of the structure with inertial damping within the pile group system.

8. Case Studies [2].

Table 1 presented a list of structures that were retrofit by Lizzi. Table 2 presents a list of selected structures with the particulars of the retrofit. The number after the structure name refers to Figure 15, indicating the geographic location of the structure. Two case studies are presented after as examples of the method.

Table 2. Partial List of Structures Retrofit by Lizzi with Particular Retrofit Measures.

STRUCTURE	RETROFIT MEASURES.
The Church of St. Ferdinando, Alvignano Caserta (5)	Overall strengthening. Roman brick. Windows. Arches. Removal of prior fixes. RRP / Foundation work.
The Amalfi Cathedral (7)	Complete structure retrofit. Arches. Open field strengthening. Out-of-plane. Edge beam – tension restraint.
The Church of St. Silvestro in Capite, Rome (3)	Complete structure retrofit. RRP + IRM. Vault barrel ceiling in nave of church.
The Church of San Lorenzo Maggiore, Naples (6)	Removal of prior retrofit. Back to “original” aesthetics. Complete structure retrofit. Strengthening of columns.
The Church of St. Andrea della Fratte, Rome (4)	Complete structure retrofit. RRP + IRM. Continuity of structure. Dome retrofit. Dome cinch band – edge beam.
Ponte de Tri Archi (2)	Complete structure retrofit. RRP + IRM.

	Bridge. Worked in-situ. No coffer dam. Erosion distress.
Nicosia's Cathedral (8)	Seismic intervention. Complete structure retrofit. RRP + IRM. Damping of EQ loading by RRP.
The Cathedral in Spilimbergo (1)	Post EQ work. EQ concepts of retrofit listed by Lizzi. Structure was loading during and after retrofitting.

IRM = Internal Reinforcement Method ("reticulo cementato").
RRP = Reticulated Root Piles ("pali radice").
EQ = Earth Quake.

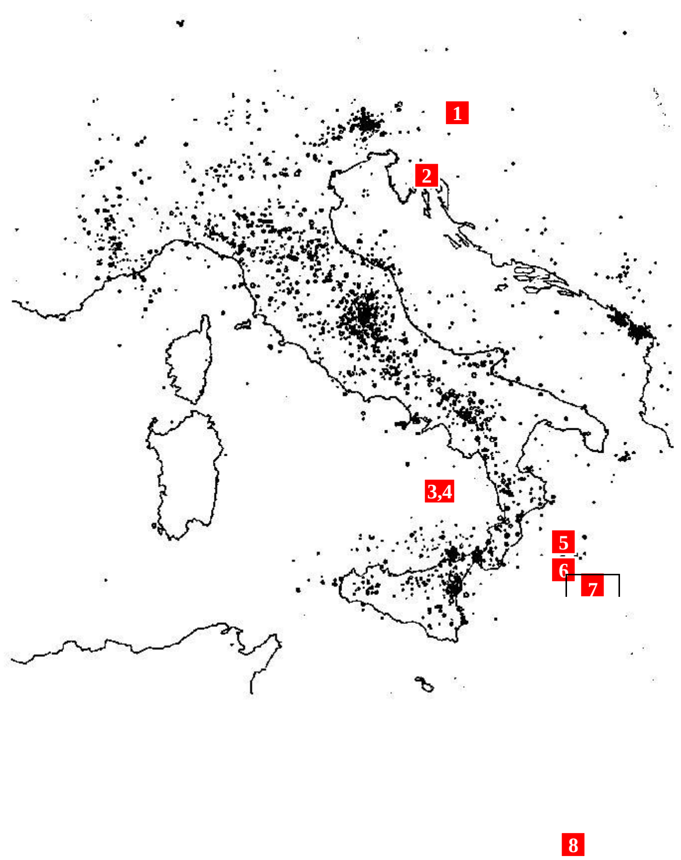

Figure 15. Earthquakes MM4 and Greater. Years 1000 to 1980 AD[12].
Location of Several Structures Retrofit by Lizzi.

Spilimbergo Cathedral.

The Spilimbergo Cathedral is located in the Friuli region of northern Italy. The seismicity of this area has a long history. In May and September of 1976, this area was devastated by large earthquakes and subsequent aftershocks. The Spilimbergo Cathedral is a XII century structure that was heavily damaged by the initial earthquake. Lizzi was asked to come and intervene after the first seism.

The structure is somewhat irregular in shape with the main walls not parallel to the roof ridge line. Both ends of the structure are not square to the ridge line, the main entrance being more closely aligned than the back wall. The back wall was constructed perpendicular to the two main side walls, see Figure 16.

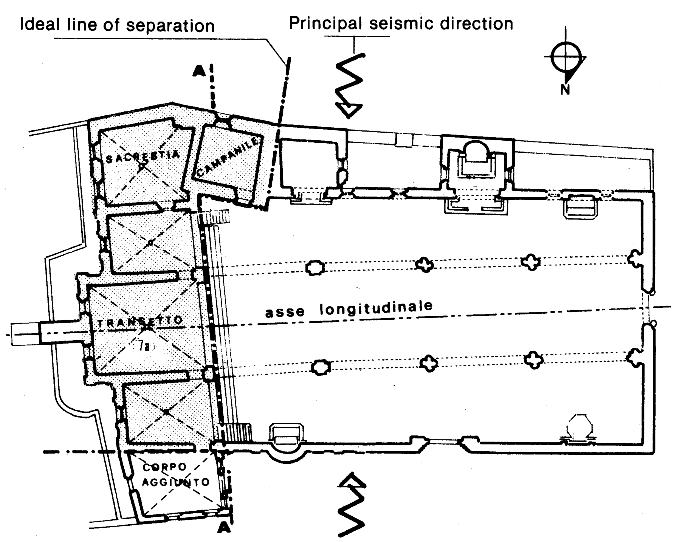

Figure 16. Plan View of the Spilimbergo Cathedral. Friuli Region, Italy[2].

The principal direction of the seismic traveling waves was in a north-south direction, thus loading the cathedral normal to the two main walls. The thinness of these two walls reduced the inertial loading and out-of-plane movements. The walls were shaken and moved permanently out-of-plumb. The main roof timber beams, with alignment in the direction of the earthquake, became battering rams pounding at the top of the wall. The beams were partially unseated at the end of the shaking. The bell tower (the campanile) and transept performed well, with minimal torsional loading because of the direction of seismic loading.

When Lizzi arrived at the cathedral, the displacements of the main longitudinal walls were in a precarious state of stability, see Figure 17. The three transverse shear walls provided enough restraint to prevent collapse during the first main event. Yet, the soundness and the stability of the structure were at great risk to any further seismic action. The high probability of seismic aftershocks necessitated an intuition and understanding of the dynamic action of the retrofitted structure. The intervention began from this experience base for Lizzi. The retrofit measures for the cathedral were, first and foremost, to avoid adding any structural mass to the upper zones of the structure. This included having minimal interaction with the roof structure. Secondly, the structure required reticulo cementato reinforcement to resist the bending tensile stresses in the out-of-plumb walls and columns. Corner and wall connections were reinforced as previously described. Also, the tower was reinforced along its length. Third, the roof timber beams had to be improved for connectivity to the walls. And lastly, the original spread footing foundation system was consolidated with grout and reinforced with reticulated micropiles in the cross-over nodal pattern along the length of the main walls. It should be noted that several aftershocks did occur, loading the structure; some during the restoration and several larger aftershocks after the consolidation.

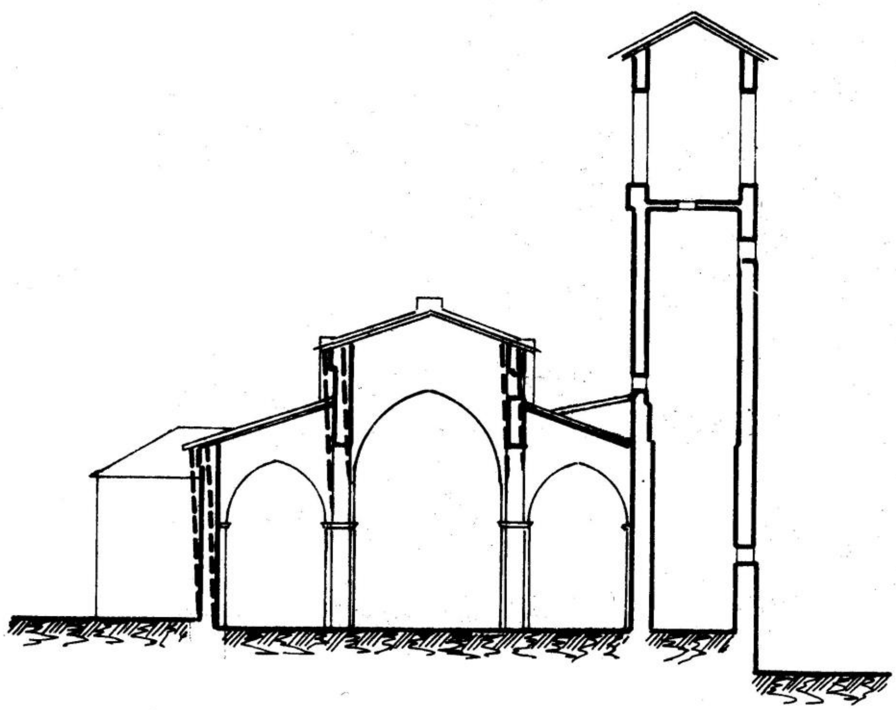

Figure 17. Spilimbergo Cathedral. Post Earthquake Condition [2].

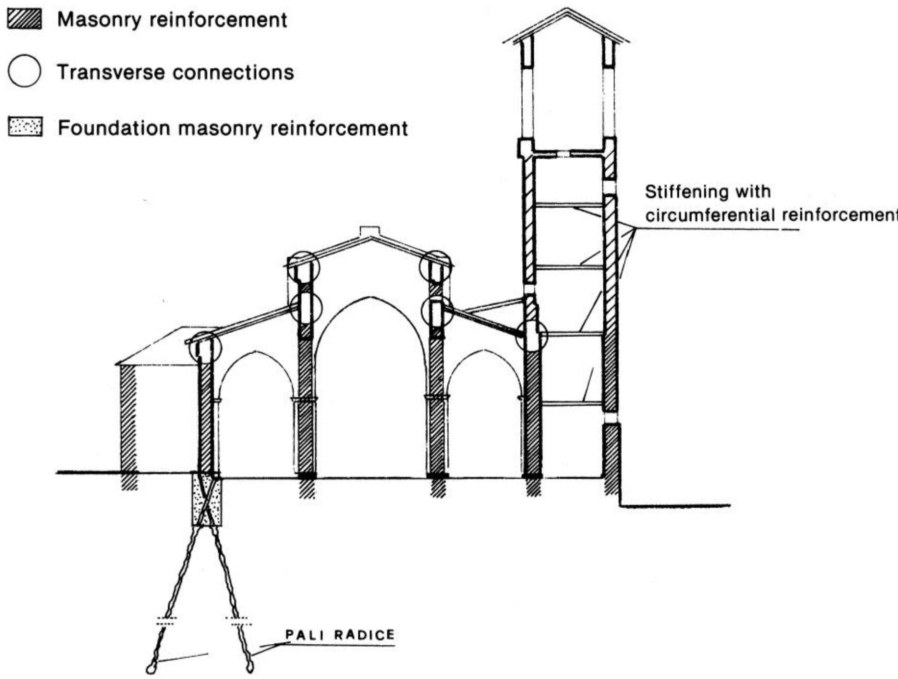

Figure 18 Spilimbergo Cathedral. IRM and RRP Retrofit Scheme [2].

Figure 19. Spilimbergo Cathedral. Retrofitted Structure [2].

The Three Arches Bridge.

The “Three Arches Bridge” was constructed on the Rio di Cannaregio Canal in Venice, Italy in 1688, see Figure 20. The bridge was designed by Antonio Tirali[13], and was the winner of a design competition with six other competitors[14] . 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”[15]. It is the only three arch bridge in Venice today.

The general area of Venice is underlain by lightly over-consolidated (LOC) Holocene fluvial deposits. The seismic risk for this area is minimal with different researchers assigning values from “no risk” to “slight risk with an expected ground acceleration of approximately 4 to 8% g horizontal ground acceleration”.

FIGURE 20. Three Arches Bridge. Venice, Italy

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. 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 equal to 0.1. This is in the optimum range for arch performance [3].

Due to the large increase of boat and gondola traffic on this canal in modern times, the erosion of the support soils rapidly increased, leading to incompatible movement between the arches and the parapet walls. This differential movement caused tension cracking to occur in the westerly end of the south parapet wall. The amount of cracking brought enough concern to the officials of Venice, that they were considering the demolition of the structure. Lizzi had been involved with other structures in Venice and also 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.

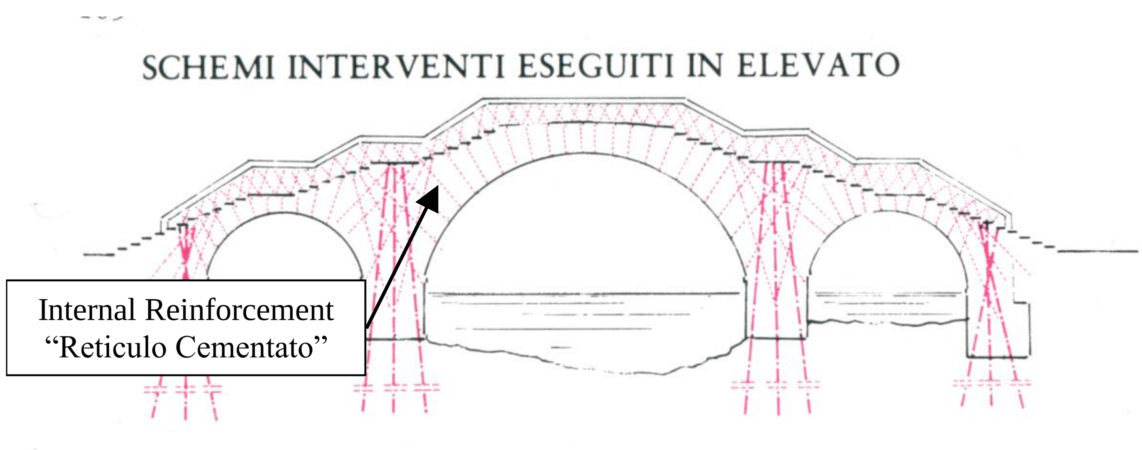

FIGURE 21
The IRM Retrofit Scheme of the Three Arches Bridge [2].

The overall goal of the IRM retrofit scheme was to create a structure that performed as an integrated continuous unit. There were several components that needed individual attention. First, the foundations were retrofit with reticulated micropiles from the bridge walkway. Then the parapets needed 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. Since there was relative movement between the bridge arches and the parapets walls, the parapet walls were tied to the arch through horizontal reticulated reinforcement. Once the IRM and foundation work had been completed, a complete repointing of the structure was conducted.

9. Non-Destructive Analysis in Structure Investigation and Design.

Design of an IRM retrofit scheme (bar placement and orientation as well as grout type and pressure) is dictated by the in-place material conditions and the construction configuration and lay-up of the masonry element. The internal material and construction conditions can effectively and efficiently be characterized with combined applications of various nondestructive test methods. The tests serve to compliment each other in the type of information they provide.

Stress wave testing such as direct-transmission testing can give initial indications if the element is continuous throughout its thickness or if zones of rubble infill or collar joints exist internally. If the element is determined to not be continuous, impact-echo testing (with an established velocity value from indirect-transmission testing) can be conducted to establish thicknesses of outside wythes or layers. For continuous elements of known thickness, lateral variations in average stress wave velocities (through the element) obtained with the use of both direct-transmission and impact-echo methods can give indication of location and extent of interior zones of deterioration, voids or construction flaws and can be correlated to lateral variations in average material stiffness. Impact-echo can also be used to reveal depths to voids or flaws. The Spectral-Analysis-of-Surface-Waves (SASW) stress wave method may be used for further evaluations to characterize general material stiffness variations with depth (into the element) depending on the element's overall thickness. Other types of nondestructive methods such as infrared thermography or radiography can be used to verify void or flaw locations established by the stress wave methods. Stiffness verifications and strength correlations can be made with the use of other in-place methods such as probe penetration, anchor pull-out, flat jack testing or load testing or by select sample retrieval and subsequent laboratory strength testing.

10. Summary.

An overview of the structural retrofitting technology for historic buildings and bridges, as developed by Dr. Fernando Lizzi, is presented in this paper. Also, Lizzi's philosophy of intervention for strengthening historic structures was presented. The techniques can be used for static restoration and for seismic retrofitting. Discussion of seismic failure modes of non-reinforced masonry and non-reinforced stone structures is presented as a basis for the understanding of the retrofit components. The types of retrofit components, when integrated in a complete system strengthening, form an internal frame system which is installed within the existing structure fabric. This technology is an engineered integrated seismic system that includes foundation retrofitting (RRP) with structural retrofitting (IRM). Lastly, a discussion of the use of non-destructive testing technologies for the design of the IRM reinforcement is presented.

[1] Lizzi, F. '"Pali radice' (root piles) and 'reticulated pali radice'", Underpinning, S.Thornburn and JF Hutchison, Editors, Surrey University Press, 1985, pp. 84-151

[2] Duryea, E, et.al., "The Effects of the San Francisco Earthquake of April 18th, 1906, on Engineering Constructions", ASCE Transactions, Vol. LIX, No.1056, May 1907, 208-329

[3] Lizzi, F, The Static Restoration of Monuments, Sagep Publishers, Genoa, 1982, 142

[4] Heyman, J, The Masonry Arch, Ellis Horwood Limited Publishers, 54-55

[5] Bruce, DA, and I Juran, Drilled and Grouted Micropiles: State-of Practice Review, Volume II: Design. Report No. FHWA-RD-96-017. FHWA, U.S. Department of Transportation, 1997

[6] Kulhawy, FH & JA Mason, " Use of Reticulated Micropile Groups for Bridge Foundations" Proc., 4th Natl. Workshop on Bridge Research, Ed. IG Buckle & IM Friedland, Buffalo, Jun 1996, 101-106. (Natl. Center for Earthquake Eng. Research, Buffalo)

[7] Kulhawy, FH & JA Mason, " Use of Reticulated Micropile Groups for Bridge Foundations" Proc., 4th Natl. Workshop on Bridge Research, Ed. IG Buckle & IM Friedland, Buffalo, Jun 1996, 101-106. (Natl. Center for Earthquake Eng. Research, Buffalo)

[8] Lizzi, F, "Durability for the man-made constructions.", Geotechnical Aspects of Restoration Works on Infrastructures and Monuments, Ed. Balasubramaniam, A.S., et al pp. 163-174

[9] Kulhawy, FH & JA Mason, " Use of Reticulated Micropile Groups for Bridge Foundations" Proc., 4th Natl. Workshop on Bridge Research, Ed. IG Buckle & IM Friedland, Buffalo, Jun 1996, 101-106. (Natl. Center for Earthquake Eng. Research, Buffalo)

[10] Terzaghi, K., "The Science of Foundations- Its Present and Future" 1927, ASCE Transactions, Paper No. 1704

[11] Personal conversation with Lizzi, Washington, DC, 1991.

[12] Earthquake Reference.

[13] Bassi, E., Architettura del Sei e Settecento a Venezia, Edizioni Scientifiche Italiane, Napoli, 1962, 269-295

[14] Puppi, L. and G.Romanelli, Le Venezie possibili. Da Palladio a Le Corbusier, Electra Editrice, Mailano, 1985, 116-118

[15] Jogna, L. F. Lizzi, and G. Machhi, 1983, "Structural Restoration of "3 Archi" Bridge. Rio di Cannaregio, Venice", IABSE Symposium Venezia, 1983, Poster Sessions, pp.20-21

[16] Lizzi, F, "The consolidation of the Leaning Tower of Pisa and the preservation of the Venice Lagoon", Geotechnical Aspects of Restoration Works on Infrastructures and Monuments, Ed. Balasubramaniam, A.S., et al pp. 17-33