SEISMIC DESIGN CONCEPTS AND ISSUES FOR RETICULATED MICROPILE FOUNDATION SYSTEMS

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ABSTRACT

The reticulated micropile foundation system appears to have inherent seismic resistance qualities superior to those of regularly spaced vertical foundation systems. The encasement of the supporting soils within the pile group provides several modes of resistance to seismically induced lateral loading. The first is the inclusion of the gravity mass of the interior soil to provide resistance to overturning. This is mobilized by the axial frictional resistance alongside the micropile shaft and the horizontal clamping action of the reticulated pile configuration. The second is the large passive strain wedge that is developed by both the encased soil mass and the exterior adjacent soils. All of these modes of resistance are developed because of the unique interaction between the reticulated micropiles and the encompassed and surrounding support soils. Due to the geometric configuration of the micropiles, the piles form a strongback from which the soil forms an arch; pile to pile. This configuration can be thought of as a completely closed surface, a soil quilt, wrapped around the piles. The system response engages a zone of soil larger than the pile group itself. Thus, the reticulated configuration provides resistance to seismic loading greater than the sum of its parts.

BASIC CONCEPTS AND METHODS OF RETICULATION FOR INTERNAL REINFORCEMENT OF STRUCTURES AND SOIL

Dr. Fernando Lizzi, the inventor of the reticulated configuration, realized the unique interaction provided by this system[1]. Lizzi, both a structural engineer and a geotechnical engineer, utilized this technology on numerous structural retrofits of historic monuments, towers, buildings, and bridges throughout Europe and in the Middle East[2]. These retrofit measures were of both the substructure, i.e., the foundation, and the above ground structure.

The general idea of the structural retrofit for the monument, building, or bridge is to drill and bond a regularly patterned network of small diameter reinforcement steel within the confines of the existing structure at critical locations. With this technology, for the majority of structures, there is no need to build additional framework within the original structure. The method of adding an additional frame, which has been the predominant method of seismic retrofit and upgrade in the United States, creates a situation where the original structure becomes the seismic inertial load on the added framework. Whereas, with the reticulated internal retrofit and strengthening, the original structure works for itself. This is a labor intensive methodology. And yet the original aesthetic of the structure is maintained. On one of the older structures that Lizzi had worked on, he removed gravity mass buttresses that were installed several hundred years ago when the "original" structure was reinforced[3]. See Figure 1, *Bari, Italy. Restoration of the Old Town. Strengthening of the old walls in order to remove the buttresses.* That is, the original retrofit took place over several hundred years ago. In fixing the building, Lizzi removed buttresses that were applied to the original structure, approximately 200 years old at the time of the first retrofit. The structure now performs wonderfully with the internal reinforcement and without the buttresses. Lizzi has developed general details for virtually every structural component. These details include wall panels, corners, columns, arches, and corbels, to name some of the general solutions that he has provided[4].

The retrofit of the substructure involves installing small diameter piles in a reticulated configuration. Think of this geometry in similitude to the woven Chinese finger cuffs. These are the woven tubes of fiber that when slipped over your finger will engage and lock onto your fingers when they are pulled apart. This analogy is also appropriate for the superstructure retrofit. The reticulated pile group will engage and lock the encased soil mass when loaded either vertically or horizontally. The construction installation of the micropile starts up in the superstructure and is drilled down and through the base of the walls of the structure and then into the soil mass. Thus, the axial load transfer from the superstructure is developed over an appropriate development length within the base of the structure itself. Except for special cases, there is really no need to add additional material to the base of the structure or the pilecap.

One of the key features of this technology is the smooth and gradual change of system stiffness from the base of the foundation to the top of the superstructure. There are no hard spots in the retrofitted structure to act as a stress riser. Another key feature of this method is that the retrofit and strengthening are accomplished without changing the original structure aesthetic. What was originally a non-reinforced masonry or non-reinforced stone structure has a network of steel reinforcement installed internally in the structural components to resist tensile stresses. These tensile stresses could be seismically induced, induced due to differential settlement caused rotation, or due to some other lateral loading. The reticulated foundation system provides the resistance framework to the seismically induced inertial forces for the supporting soils. Due to the slenderness of the structural foundation elements, the reticulated system is relatively compliant to seismically induced horizontal displacements. The axial strength and stiffness can be designed for the most stringent settlement criteria. And at the same time the reticulated network confines the encased soils, similar to spiral confinement steel in a concrete column. The encased soils "form a unitary soil / pile structure; practically a block of reinforced soil.[5]"

In terms of limiting displacement for the soil-pile system, Lizzi points out that it is the brittle rupturing[6] of the soils at which point that the encasement of the soil would fail. At this stage the soils would flow through the reticulated configuration in the upper zones where there were large displacements. The foundation system would then be restrained in the deeper zones of the foundation and have effectively an unsupported pile length above the horizontal failure plane. It must be remembered that the structural configuration of reticulation is self-protecting under lateral loading. And simple Euler buckling criteria at this point would be inappropriate. The moment couple that is formed within the reticulated pile group is well distributed to all piles within the group. Thus, any potential overloading of an individual element is redistributed to adjacent members. The basic reticulated geometric configuration is extremely forgiving, and extremely resilient.

A CONCEPTUAL COMPARISON OF SEISMIC DESIGN CRITERIA AND ESTIMATED PERFORMANCE OF VERTICALLY INSTALLED PILE GROUPS WITH THE RETICULATED MICROPILE GROUP SYSTEM.

VERTICALLY INSTALLED PILE GROUPS

Caltrans Seismic Design Methodology for Foundations.

The Caltrans design methodology for foundations is mainly a codified process. For the majority of structures in the California inventory, the foundation design process has been directed by a specified axial and lateral strength and stiffness criteria which for the most part is independent of soil type. The exceptions are for soft soils, i.e., soils with a standard blow count less than ten ($N < 10$), and for seismic loading. These special conditions are addressed by the appropriate specialists at the Caltrans Engineering Service Center. Some comments are provided below to help in the assessment and interpretation of the Caltrans codes and specifications. As with all state and federal agencies, the particular conditions for a specific project must be addressed by the agency in charge.

The current typical design of the foundation system has been separated into two disciplines; the geotechnical engineering approach and the structural engineering approach. This is typical for most projects today. Not that it is "the correct way" for this design process. In this era of specialists, the design process has been broken down into components. For civil engineering, the design of complete building structures has been broken down into a multitude of engineering specialties: civil / site development, geotechnical, and structural, in addition to all of the mechanical, electrical, environmental engineering services that are applied to the complete structural design. This appears to be true for all engineering disciplines. And for the seismic design of foundations, this is separating the system at the critical location. The decoupling of the foundation from the superstructure in dynamic analyses is missing a basic feature of the complete structural system. The interaction between the foundation and the superstructure during an earthquake is a unique exchange of energy, alternating between the driving force and the inertial force. In the modeling process, it is imperative to capture this interaction. But by the separation of the geotechnical from the structural engineering, an incomplete system is modeled. It must be remembered that the seismic energy is delivered to the structure through the foundation. The seismic wave travels through the earth and loads the structure from the foundation up. Thus, the perspective for the geotechnical engineer starts from the soil, then into the foundation and then up into the superstructure. For the structural engineer, the perspective is exactly opposite. They look at the system as though it is the superstructure that delivers the seismic energy to the system. The Caltrans methodology, in this case, is no different than the current state of practice.

Geotechnical Engineers Approach.

The seismic design methodology utilized by the Caltrans geotechnical staff is mainly directed by the "Caltrans Interim Seismic Design Criteria for Pile Survivability[7]". It is a combination of suggested methods and techniques for the analyses of pile foundations under seismic loading. An overview of the process is as follows.

1. Compute free field soil displacement. For non-liquefiable areas, it is suggested to use Total Stress Analysis with *Shake91*. For liquefiable sites use effective stress analysis and the *Sumdes* program. Compute time-history displacement and motion using the appropriate program just listed. The idea is to obtain two peak loading conditions: 1) Peak uni-directional soil strain, and 2) peak bi-directional strain.
2. For the pilecap: Compute the soil resistance loading curves, i.e., the p-y and t-z curves. It is suggested to use the Reese-Matlock[8] cyclic design criteria. This design criteria includes the cyclic degradation of the soil modulus. Also it is suggested to utilize a group reduction factor of 1.0.
3. For the pile: Use the Reese-Matlock criteria (cyclic) and the use the following group reduction factors: 1) Soft clays and liquefied sands: GRF = 0.65, 2) Sands / stiff clays - use GRF = 0.35 at depths less than 25 pile diameters and use a GRF = 0.65 at depths greater than 25 pile diameters. For liquefied sites, use a bi-linear p-y curve, (see the appendices in the Caltrans document).
4. The t-z curve are defined by the Federal Highway Administrations methodology described in the "Soils and Foundations Workshop Manual - Second Edition"[9].
5. Compute dynamic pile axial and shear loads. There are two suggested cases here. One is to have a full shear loading with a factored dead load (1.4 * D.L.) applied to the top of the pile. The other is to have 25% of the full shear plus the factored dead load (1.4 * D.L.). This case is analyzed for the two peak shear loading just listed.

Structural Engineers Approach.

The design philosophy of Caltrans for bent and pier foundations is succinctly delineated in the following paragraph.

BDS Commentary 3-17.

3.21.7 October 1989, Seismic Design of Bent and Pier Foundations.

The foundation design forces specified are consistent with the design philosophy of maintaining structural integrity under high seismic loads. The *recommended design forces are the maximum forces that can be transmitted to the footing by plastic hinging of the column. The probable plastic moment represents the best estimate of the largest moment that can be applied to the foundation and should be utilized as the ultimate design moment on the foundation whenever the ARS (Acceleration Response Spectra) moments are larger than the plastic hinging moment. The ARS elastic design forces are considerably greater that the plastic hinging forces. In cases where architectural considerations govern the design of a column the ARS elastic design forces may be less than the forces resulting from column plastic hinging. See commentary 3.21.8, and 8.16.4.4 for a detailed description of the ultimate forces. In some cases Group I vertical forces may govern the foundation design over plastic hinging.*

- BDS 4.3.3 Jan.1993 Page 4-2 Piles - Design Loads.

4.3.3.1 The design loads for piles shall be according to Article 4.3.4 Piles shall be designed to carry the entire superimposed load, no allowance being made for the supporting value of the material between the piles.

This specification penalizes or dismisses the use of the encased soils of any kind of pile group for additional support. This makes sense for vertical pile groups, but penalizes the reticulated micropile system mechanism. For agencies such as Caltrans, they must wait for conclusive laboratory and / or field testing to rationally prove the engagement of the encased soil mass as part of the resistance mechanism

- BDS 4.3.4 Jan.1993 Page 4-2 Piles -Load Capacity of Piles.

4.3.4.1.1 The design load on a pile shall not be greater than its load capacity as determined from the minimum of the following cases:

Case A: The capacity of the pile as a structural member.
Case B: The capacity of the pile to transfer its load to the ground.
Case C: The capacity of the ground to support the load from the pile or piles.

4.3.4.1.2 The values of each of these cases shall be determined by making subsurface investigations or tests and by referring to all available information. Consideration shall also be given to:

(1) The difference between the supporting capacity of a single pile and that of a group of piles.
(2) The capacity of the underlying strata to support the load of the pile group.
(3) The effects on adjacent structures of driving the additional piles.
(4) The possibility of scour and its effect.
(5) The transmission of forces from consolidating soils.

4.3.4.6. Uplift

4.3.4.6.1 Friction piles may be considered to resist an intermittent but not sustained uplift. Resistance may be equivalent to 40 percent of both the allowable and the ultimate compressive capacity, except for seismic loads it may be equivalent to 50 percent of the ultimate compressive load. Adequate pile anchorage, tensile strength, and skin friction must be provided. In no case shall the uplift exceed the weight of material (buoyancy considered) surrounding the portion of the pile.

4.3.4.7 Group Pile Loading. The capacity for a group of piles shall be determined by an analysis of subsurface conditions.

This task is accomplished by utilizing one of several commercial programs available. See the Federal Highway Administration document "Drilled and Grouted Micropiles: State-of-Practice Review. Volume II: Design,[10] for suggested methods.

4.3.4.8 Lateral Resistance. Lateral resistance of piles fully embedded in soil with a standard penetration resistance value, N, of 10, and with a ¼ inch maximum horizontal deflection under service load shall be:

CIDH Concrete (16")	*13 kips*
Driven Concrete (15")	*13 kips*
Driven Concrete (12")	*5 kips*
Steel (12" or 10" flange)	*5 kips*
Steel (8" flange)	*4 kips*
Timber	*5 kips*

The lateral resistance of piles not within these criteria shall be determined by geotechnical analysis and structural adequacy of the pile.

At bent and pier footings the number of piles required for lateral pile resistance shall not be governed by Group VII loads (seismic loads).

The horizontal component of a battered pile's axial load may be added to the lateral resistance.

The seismic design of bridges has been rigorously questioned and investigated, with codified results and design guidelines, by the Caltrans staff[11] and formal academic researchers. Zelinski and Yashinsky, seismic specialists at Caltrans, have investigated the ultimate capacity and performance of numerous piles types to their limit state. See Table 1, *Ultimate Lateral Resistance Capacities for Various Pile Types.* These tests were conducted at the former Cypress Structure in Oakland, California. The tests are documented by Abcarius[12]. Zelinski recommends the following values for evaluating existing pile foundations for ultimate capacity.

Table 1.
Ultimate Lateral Resistance Capacities for Various Pile Types

Pile Type	Soil Type	Ultimate Lateral Capacity (kips / pile)	Max. Allowable Displacement (inches)	Lateral Soil Spring (kips / inch)
Steel (any size)	Dense Granular	100	3	35
Steel (any size)	Loose Granular	75	3	25
Steel (any size)	Soft Cohesive	60	2	30
Concrete	Dense Granular	40	1	40
Concrete	Loose Granular	40	2	20
Concrete	Soft Cohesive	40	2	20

It should be noted that these recommended values rely on both the pile and pilecap interaction with the surrounding soil. It should also be noted that the passive resistance due to the pilecap can only be engaged up to the maximum lateral displacement of the foundation system. After this displacement, the soil has been pushed away from the edge of the pilecap and can not be further engaged.

One other interesting point made in the Caltrans specifications is for piles founded in soils with a standard penetration index of at least 10, ($N \geq 10$). In the Bridge Design Specifications (BDS) 4.3.3, pg. 4C-2, it is stated that "the piles do not add significantly to the stiffness of the support". In other words, it is the stiffness of the soil that dominates the pile group stiffness. This implies that it is the reinforcement of the soil by the deep foundation elements that provides the foundation structural system stiffness. It seems to this author that this provision acknowledges and provides for the opportunity for the use of reticulated micropile groups.

THE RETICULATED MICROPILE GROUP SYSTEM

CURRENT DESIGN CONCEPTS.

Soil Reinforcement vs. Deep Load Transfer

Lizzi had always envisioned the reticulated root pile system to encompass and engage the surrounded soils as the major component to load resistance[13], more than the micropiles themselves. The Panorama Tower, Tokyo, Japan is an example of a new structure and foundation built with the reticulated technology. See Figure 2. One of the first projects to take advantage of this feature was a landslide stabilization. The design "took advantage" of the "gravity retaining wall" created by the reticulation. It worked very successfully. Yet as bigger and stronger became the trend for other designers, the use of reinforced soil became outdated and shelved. The general attitude towards the use of the soils as the primary load resisting element was replaced. The equipment and drilling technology advanced and load demands increased. And which came first is uncertain. Contractors started to use micropiles with a large percent of steel, with respect to the original "palo radice". The load was being taken deeper and deeper. And now it is commonplace to see only deep load transfer hybrid micropiles.

Although both the "palo radice" and the other micropile types are small diameter bored piles, their performance characteristics are very different. Again to quote Lizzi[14],

> *"Steel pipe micropiles, cemented into the soil.*
>
> It has been said before, about "Pali Radice", that their high bearing capacity, compared with their small diameter, is the most favorable characterizing element.
>
> On the other hand in a "palo-radice", as in any concrete pile, the bearing capacity has its limit in the crushing resistance of the cross section of the shaft.
>
> Therefore the tendency arose to increase the metal reinforcement to obtain more resistant sections. Finally micropiles consisting substantially of very heavy metal pipes (or structural beams), which would bear considerably high loads, were introduced. Practically they are pipes cemented into the subsoil."

The load shed characteristics are compared in Figure 3, *Load Characteristics for "Palo Radice" and Steel Micropile,* for (a) "Palo Radice" and (b) a steel micropile. The Case 2 micropile effectively and immediately sheds the applied loads such that no load reaches the pile tip. The steel micropile, i.e., the Case 1 micropile, sheds the applied loads at depth. Thus, the steel micropile will experience elastic shortening in addition to any permanent movement of the complete pile, whereas the "palo radice" will immediately transfer load to the surrounding soils with less deformation. Figure 4, *Load Settlement Comparison Between "Palo Radice" and Steel Micropile,* illustrates an actual comparison test between a "palo radice" and a steel micropile.

Elastic Deformation and Permanent Set

Because of the unique load shed characteristics of the "palo radice", it will have minimum deformation at an applied load. For the steel micropile, the deformation can be separated into two components: (a) elastic deformation, and (b) permanent set. A designer who is acutely aware of the performance characteristics of the micropile of choice, and who has an accurate description of the immediate soil profile, can engineer the load deformation characteristics into their design. These two components have been investigated by Bruce, Bjorhovde, and Kenny[15,16], who have defined the "Elastic Ratio" as the quotient of the resultant elastic deformation and the associated applied load, ($ER=\Delta_e/Q$). It is a simple indicator of the effective composite elastic modulus of the grout filled casing. This readily determined value then is used to determine the point of beginning, along the shaft of the micropile, for the load transfer.

CURRENT CAPACITY CALCULATION CONCEPTS

Since the origin of the micropile was in Europe, it is very understandable that the majority of calculation methods have come from there. From the Littlejohn and Bruce[17] Rock anchors - state of the art series to the French Recommendations Clouterre 1991[18], the various methods for calculating micropile capacities, and for these two cases, rock anchor and soil nail, all converge on the same basic question: How to account for the developed side resistance at the pile system - soil boundary. To help visualize the mechanisms of the reticulated micropile group, Kulhawy and Mason[19] have developed a language to describe the components of the system. Those components are: 1) The Node, 2) The "Soil Diamond", 3) The "Quilted System", and 4) The Reticulated Micropile Group. See Figures 5, 6, & 7.

Axial Capacity

The axial capacity of a single micropile, and of a micropile group, is influenced, in varying degrees, by some of the following items: group geometry, micropile structural materials, geologic stratigraphy, in-situ soil stress state, soil strength, installation method, and type of loading, to name some of the influencing factors[20]. Inclination is another variable in the calculation, with different response in the compression and tension cases. (Yet, the state of the art for the calculation of the axial capacity of micropiles and reticulated micropile groups, the analysis of side resistance, is an estimation via empirical methods.) Some of the empirical methods that are utilized by designers were originally developed for different installation methods, i.e. the use of driven pile calculations for drill and gravity grouted installation. Therefore, the estimation of axial capacity by these analysis methods need to have correction factors which are correlated with estimated site conditions. These are all approximation methods with questionable results. The contractor either performs proof of performance tests, or has prior experience with similar factors of influence. These methods have obviously been used with a great deal of success. But the question must be asked: Was the method of analysis which predicted the pile performance accurate? Or, does the inflated factor of safety mask the under prediction? These important questions need to be acknowledged and investigated.

The axial capacity calculations are separated into the compression and tension cases, with different soil and structural issues that need to be accounted for in the analysis of each action.

For the "palo radice", the compression component would be virtually equal to the tension component. The difference would be the relatively small amount of tip resistance, which for all intents and purposes could be ignored.

The steel hybrid micropile requires a different approach. Due to the distinct components used for developing the compression resistance verses tension resistance, the capacity calculations need to account for the different actions.

The primary calculation that is to be carried out for the compression capacity of a hybrid micropile is the side resistance developed along the entire length of the pile. This is separated for the two major parts of the micropile: the grout bulb, (grouted length), and then for the prismatic elastic length, (unbonded zone). A brief, yet interesting paper by Kishida, Horiguchi, and Murakami[21] delineates the compression resistance of micropiles with pressure bulbs, which have an effective diameter greater than prismatic section, along a length of their tips. The modeling process, based on work by Bruce, and work by Vesic, was tested against full scale micropiles with very good correlation. The evidence of increased compression performance due to the pressure injected grout bulb was very clear. This increase in capacity was substantially more than the increase in point bearing due to the larger diameter of the bulb.

The tension capacity of a pressure grouted micropile is developed mainly in the grout bulb zone. The grouting pressure has a direct correlation to the capacity[22,23]. The tension element is anchored in this portion of the pile, with the elastic elongation length extending from the grout bulb up to the top of the pile. The greater the grout pressure, the greater the compaction of the immediate soils, also the greater the clamping action onto the tension element. The elastic elongation depends upon the type of tension element; wire, strand, or bar, and the length of unbonded element. One other interesting and effective use of the tension anchor is to preload the supporting soil via compression on the pilecap. Then when the actual construction loads are applied, there will be less settlement.

Lateral Capacity

The lateral resistance of all pile types is primarily developed by the passive resistance of the adjacent soils. Side resistance is also developed due to top of pile movement and rotation, adding to the general performance[24]. For these long slender elements, the lateral loading resistance of a single micropile in comparison with a larger foundation element is proportionally less by a function of cross-sectional area and bending stiffness. There are closed form, rational analyses, such as Poulos and Davis[25], and Broms[26] for calculating an approximate lateral capacity. There are also empirical methods which have been utilized by many designers. For long, slender, and flexible deep foundation elements, another method for the modeling and calculation of the lateral capacity, is by "Subgrade Reaction Analysis"[27]. Matlock and Reese developed a generalized solution process. Later, Reese developed the computer program "COM622", and then subsequent editions up to "COM624P. Again it is emphasized that the "subgrade reaction analysis" method is an empirical approach. This analysis method will not include the soil arching effects that are developed by reticulated micropile groups.

Vertical Group Effects

The interaction of closely spaced foundation elements with one another is a complex soil structure problem. Brown, Reese, and O'Neill[28] studied the full scale effects of bi-direction, cyclic, lateral loading on a 3x3 pile steel group. Mason[29] has analyzed lateral load tests on two sets of driven pile groups that were founded in soft clay, (San Francisco bay mud). Abcarius conducted tests at the former site of the infamous Cypress Structure[30]. All tests show that driven pile foundations will experience substantial lateral stiffness degradation due to either cyclic loading (Brown, Reese, & O'Neill) or through large deflection (Mason, Abcarius).

Reticulated Group Effects

There are many publications that discuss and outline the analysis and design of vertically installed pile groups[31], yet to find information about design and analysis of reticulated micropile systems is rare. Lizzi discussed this phenomena in one of his recent papers[32], and his discussion was brief and empirical. Yet what Lizzi ultimately proposes is a concept of the "Reinforced Soil" bulb. His image of the "root ball" is developed into a soil mass acting as a unit. He provides very general equations for the analysis of stability of the group under general loading conditions. Still, these are empirical equations with a lot of latitude for interpretation.

This is a complex three-dimensional, pile-soil-pile interaction problem. See Figure 8, *Section of reticulated Micropile Group: Confined Soil Reaction Resulting From Applied Axial Loads,* and Figure 9, *Global Continuous Surface Response From Quilting Effect.* The current state of analysis has yet to be rigorously developed for this case. Another source, which discusses reticulated micropile groups, is the new reference by Xanthakos, Abramson, and Bruce[33]. In this text, Bruce provides the reader with an overview of micropile technology. His discussion includes the reticulated micropile group. There have also been some confirmation test projects to date by the French FOREVER[34] project team of Schlosser (Terrasol), Frank and Unterriner (Cermes), Mascardi[35], Herbst[36], and by Plumelle[37] (CEBTP).

Example Calculation and Comparison between Vertically Installed Piles with Reticulated Group Effects Under Lateral Loading.

A comparison between the response of a vertically installed pile group with the reticulated micropile group is estimated for an example case. The estimation for the vertical system is based upon the suggested ultimate load resistance by Caltrans for concrete piles as suggested by Zelinski. The resistance and model for the reticulated group is based upon the response of the system as a unit, i.e., as a reinforced soil mass. This example is modeled upon the axial load tests that Lizzi performed. See Figures 10, 11, & 12. He looked at the net increase in resistance to axial loading for both a vertically installed micropile group and a reticulated micropile group.

To estimate the size of the pile group, we will assume a 20 cm diameter micropile section. The section is of a drilled and grouted pile with a 20 mm steel bar in the center. This is a typical size for a micropile. Lizzi's test had a pile-to-pile spacing of seven pile diameters for the vertical piles. Thus, the pile-to-pile spacing was 140 cm. With this layout, the overall pile group diameter as 560 cm. Lizzi's test had a length to diameter ratio (l/d) of 100. At this length, the pile would definitely act as an embedded flexible element, as opposed to a rigid body element. For the reticulated pile group, there were two concentric pile groups. The outer diameter was approximately 80% of the vertical pile group diameter, at 450 cm. See Figure 13.

To estimate the capacity of the vertical pile group, the suggested value by Caltrans for the lateral resistance of a 12" wide concrete pile (40 kips / inch, 710 kN/m) was proportioned by the ratio of loaded area to the 20 cm micropile diameter.

$$P_{7"} = (7/12)^2*(40 \text{ k/in/pile}) = (0.34)*(40) = 13.6 \text{ kips/pile } (61 \text{ kN/pile})$$

With suggested reduction factors due to the pile-to-pile spacing, (GRF = 0.9), the total ultimate lateral capacity for the vertical pile group is estimated as:

$$P_{vert} = (5 \text{ piles})*(13.6 \text{ k/pile}) + (18\text{-}5 \text{ piles})*(0.9)*(13.6 \text{ k/pile}) = 227 \text{ kips } (1020 \text{ kN}).$$

For the reticulated pile group, the capacity is estimated from a passive strain wedge developed by the compete pile group system. Assuming a minimum depth for the soil wedge depth equal to the width of the pile group at the surface, i.e., H = 450 cm = 14.8'. For one foot of width of the pile group the strain wedge resistance would be:

$$P_p = ½*\gamma*H^2*K_p$$

For a $\Phi = 30°$, $P_p = 36.1$ kips /foot of width (530 kN / m)of effective strain wedge.

For the complete reticulated group, an effective width of 23.4' (14.8' + 8.6') is assumed. Thus for the group resistance:

$$P_{Reticulated} = (14.8' + 8.6')*(36.1 \text{ k/ft}) = 845 \text{ kips } (3800 \text{ kN}).$$

Thus, the overall improvement in performance would be 845 / 227 = 3.7. This would represent the low value of increase in performance for the reticulated micropile group. Again this is an estimate of performance. Lizzi's axial load test showed approximately a 30 to 120 % increase in axial performance over non-interacting vertically installed piles. We have shown that the lateral resistance could be even more.

CONCLUSIONS

Even with the work by Lizzi investigating and implementing the usage of reticulated micropile groups in seismic zones, the design methodology is still empirically and experientially based. The Japanese design of reticulated micropile groups also does not take into consideration the engagement of the complete system resistance mechanisms. It seems that with a foundation and structural system of such effectiveness, that not to take advantage of the numerous resisting components is wasting technology and investment. The performance of the reticulated system has been proven in Italy under seismic loading. We should move forward here in the United States.

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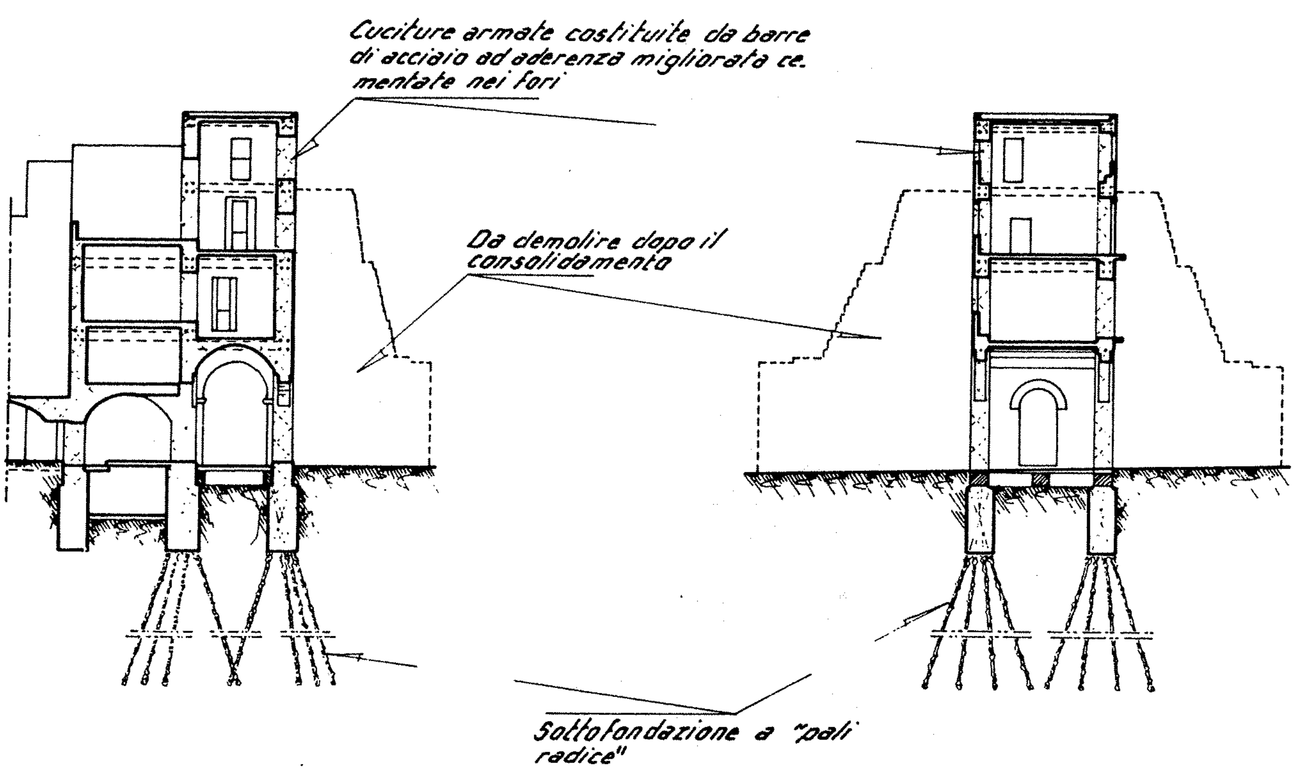


Figure 1. Bari, Italy. Restoration of the Old Town.
Strengthening of the old walls in order to remove the buttress

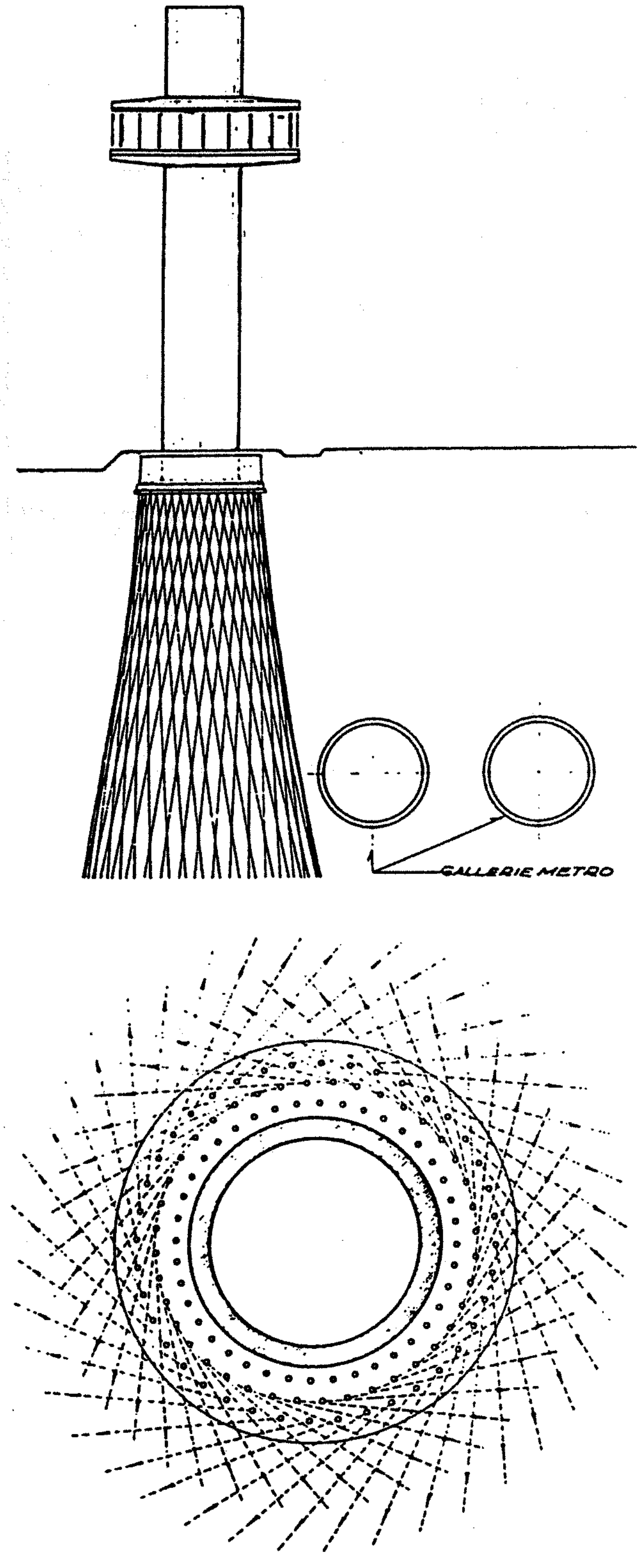


Figure 2. Panorama Tower, Tokyo, Japan

Source: Lizzi 1982

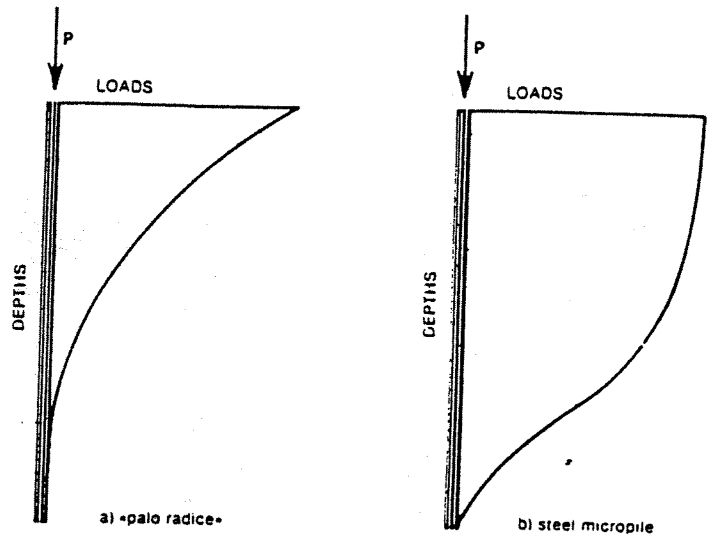


Figure 3. Load Characteristics for "Palo Radice" and Steel Micropile

Source: Lizzi 1982

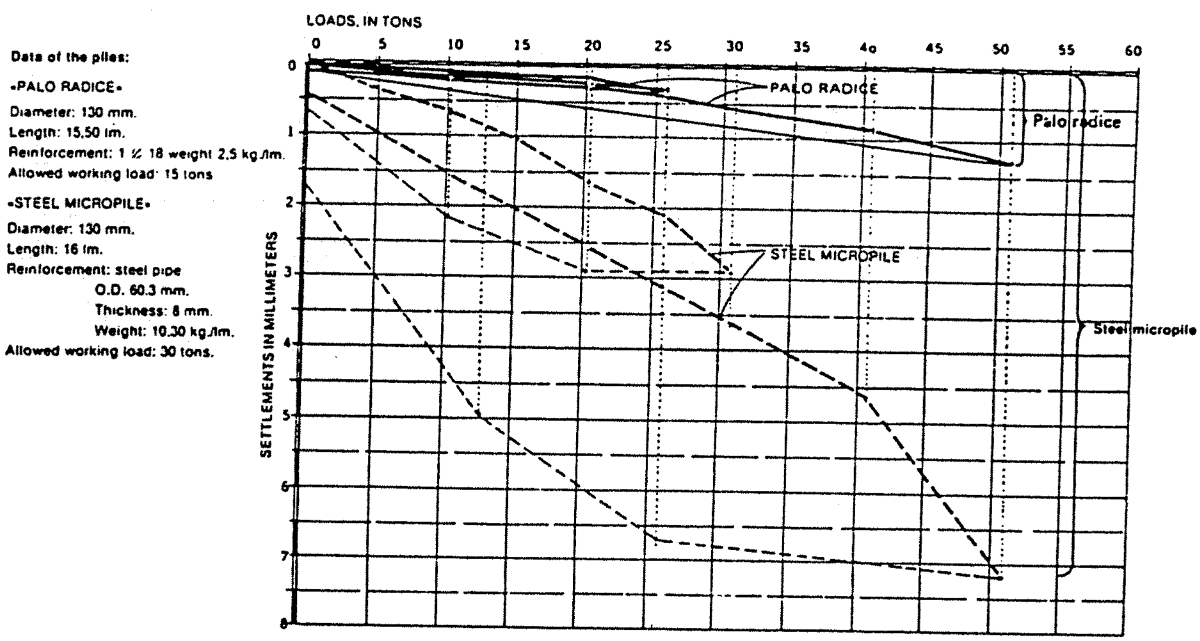


Figure 4. Load Settlement Comparison Between "Palo Radice" and Steel Micropile

Source: Lizzi 1982

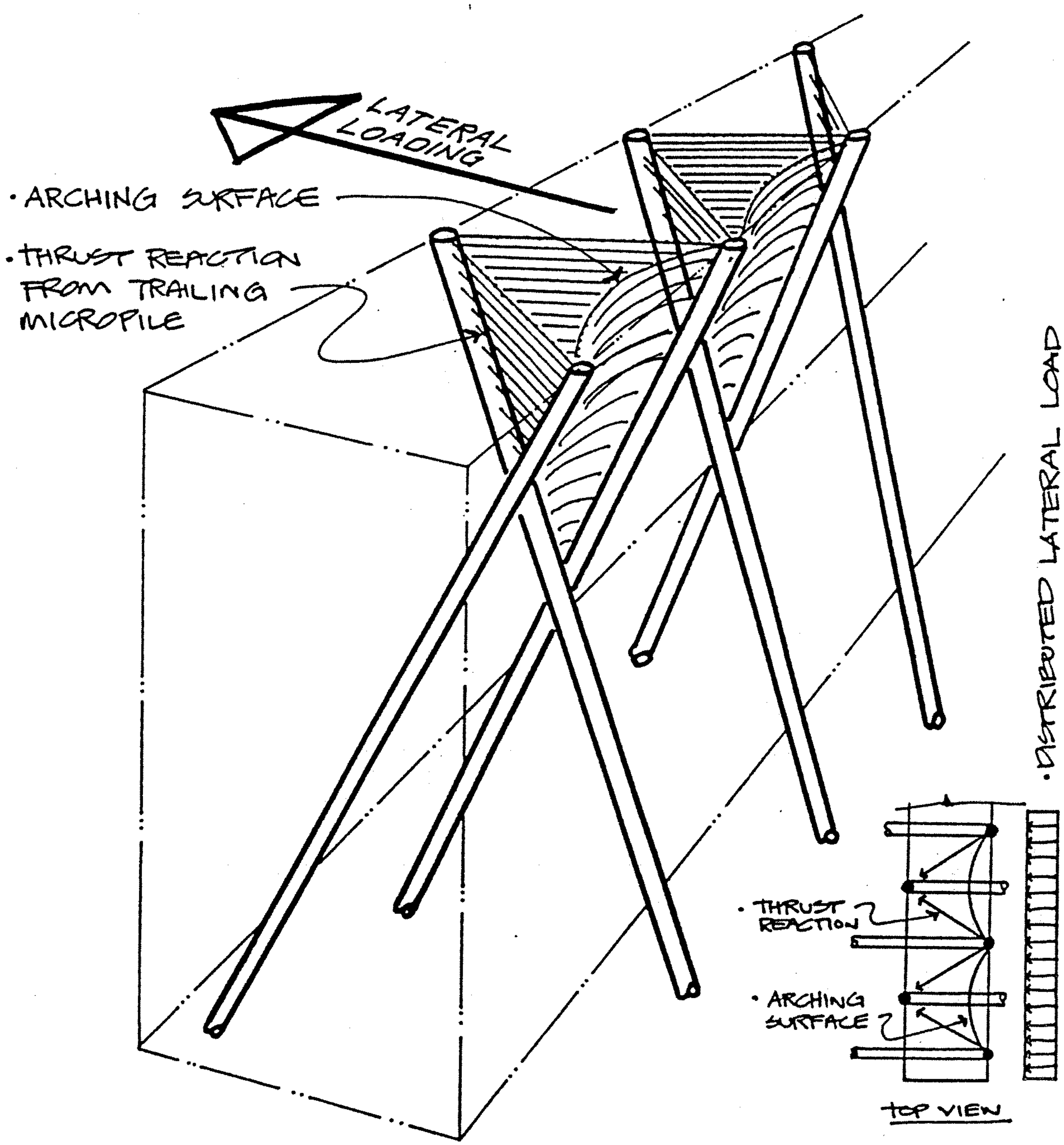
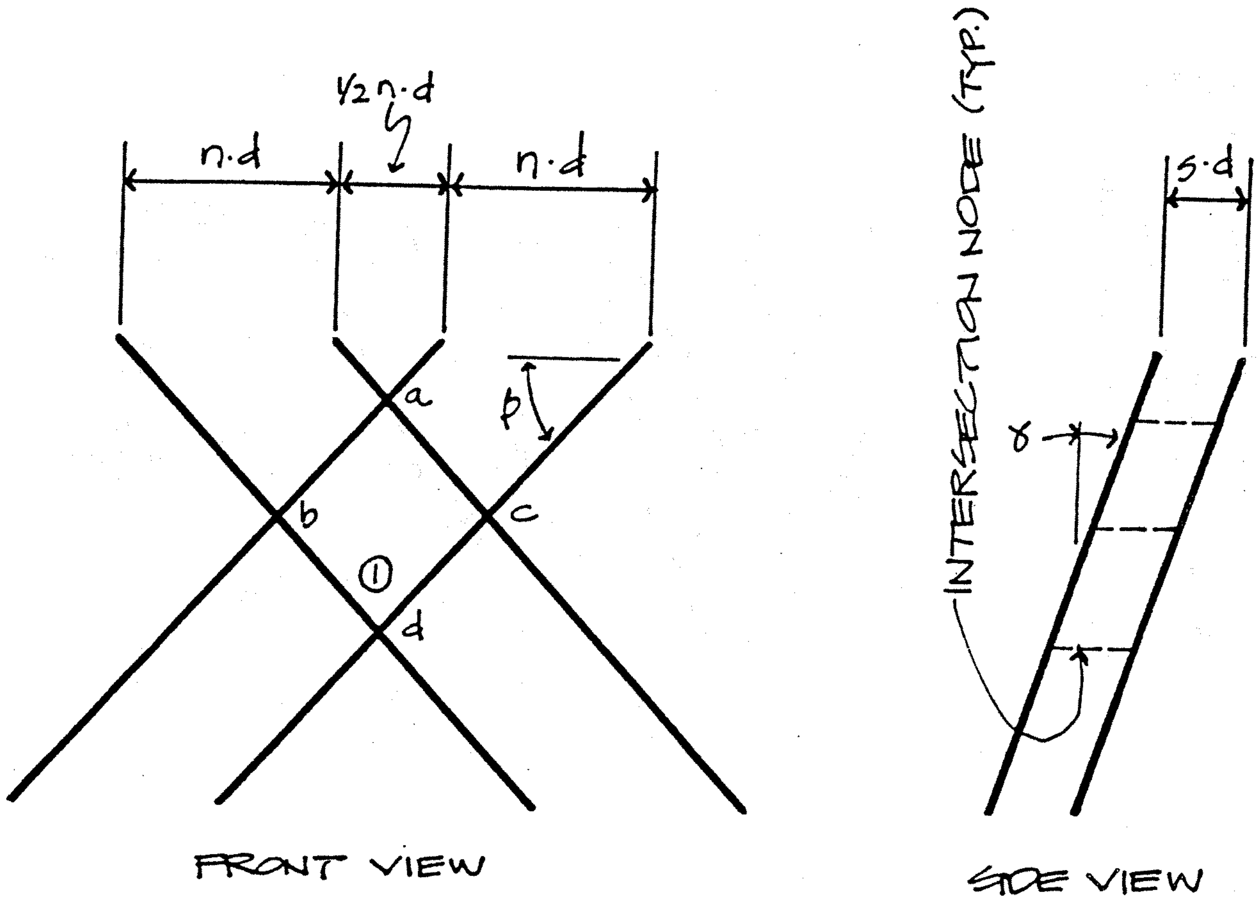


Figure 5. Laterally Loaded Reticulated Micropile Wall.



DEFINITIONS

1. d = MICROPILE DIAMETER
2. s, n = SPACING FACTOR
3. (i) = SOIL DIAMOND NUMBER
4. $a..d$ = INTERSECTION NODE
5. β = BATTER ANGLE
6. γ = INCLINATION ANGLE

Figure 6. Reticulated Micropile Group Component: the "Soil Diamond"

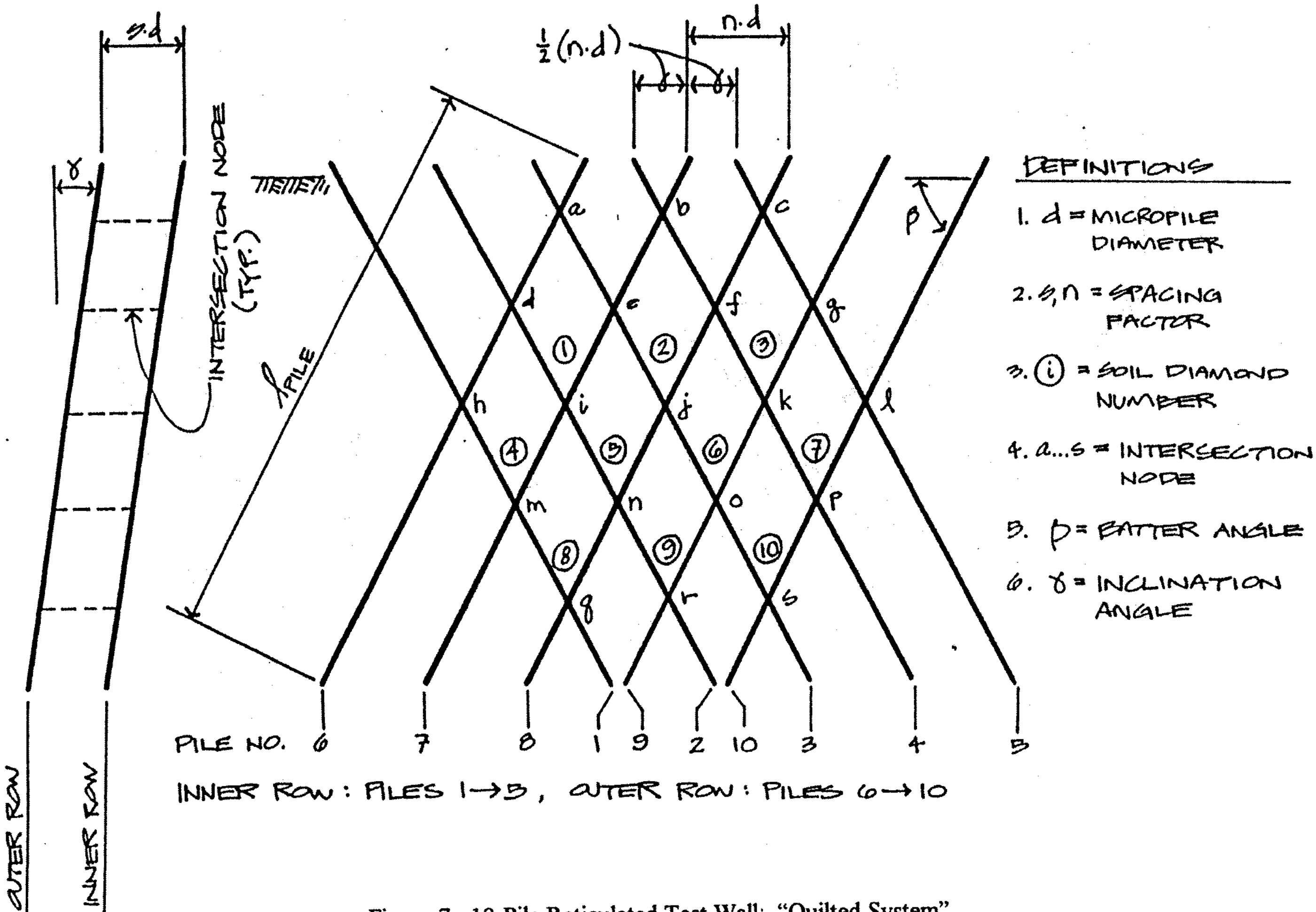


Figure 7. 10-Pile Reticulated Test Wall: "Quilted System"

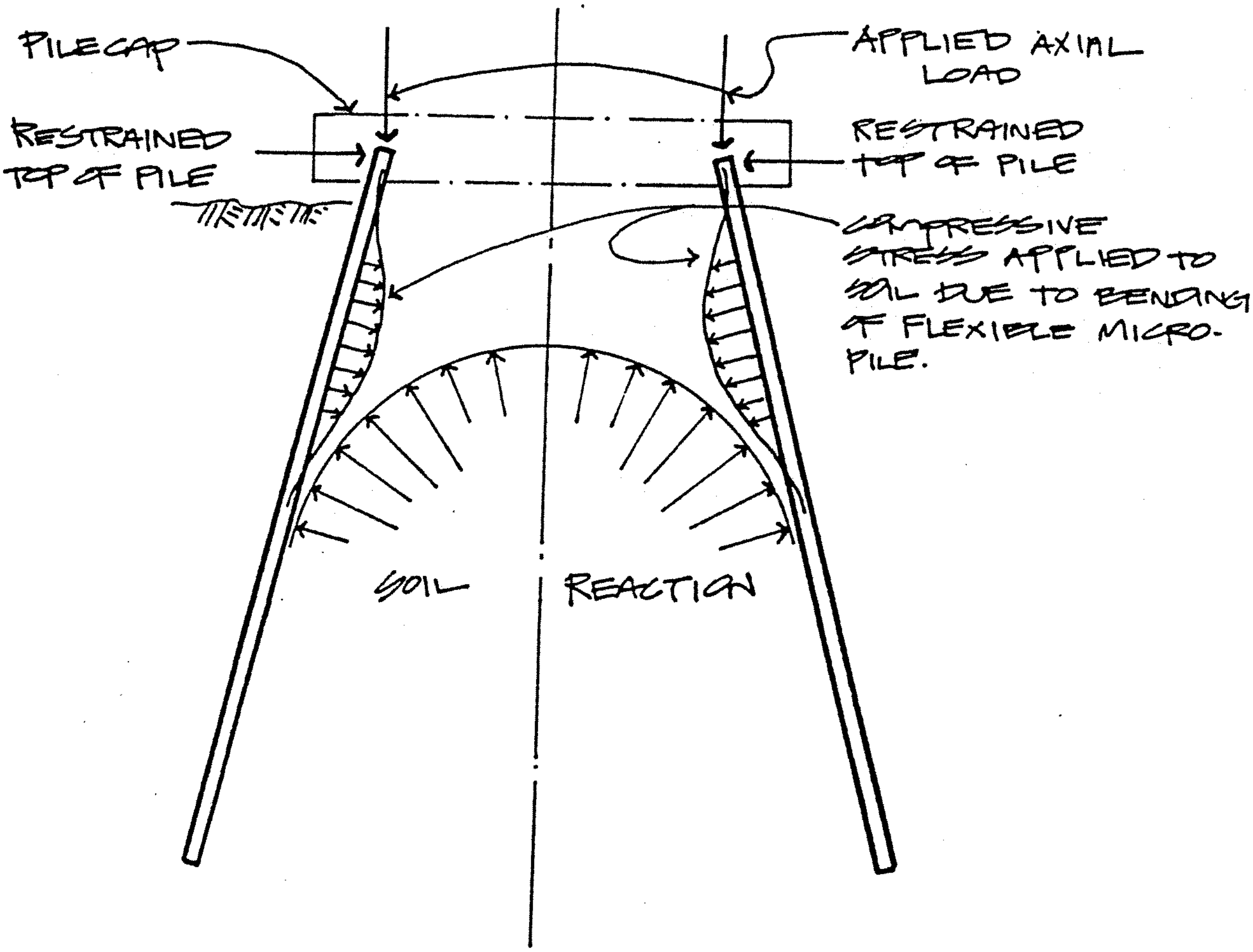


Figure 8. Section of Reticulated Micropile Group: Confined Soil Reaction Resulting From Applied Axial Loads

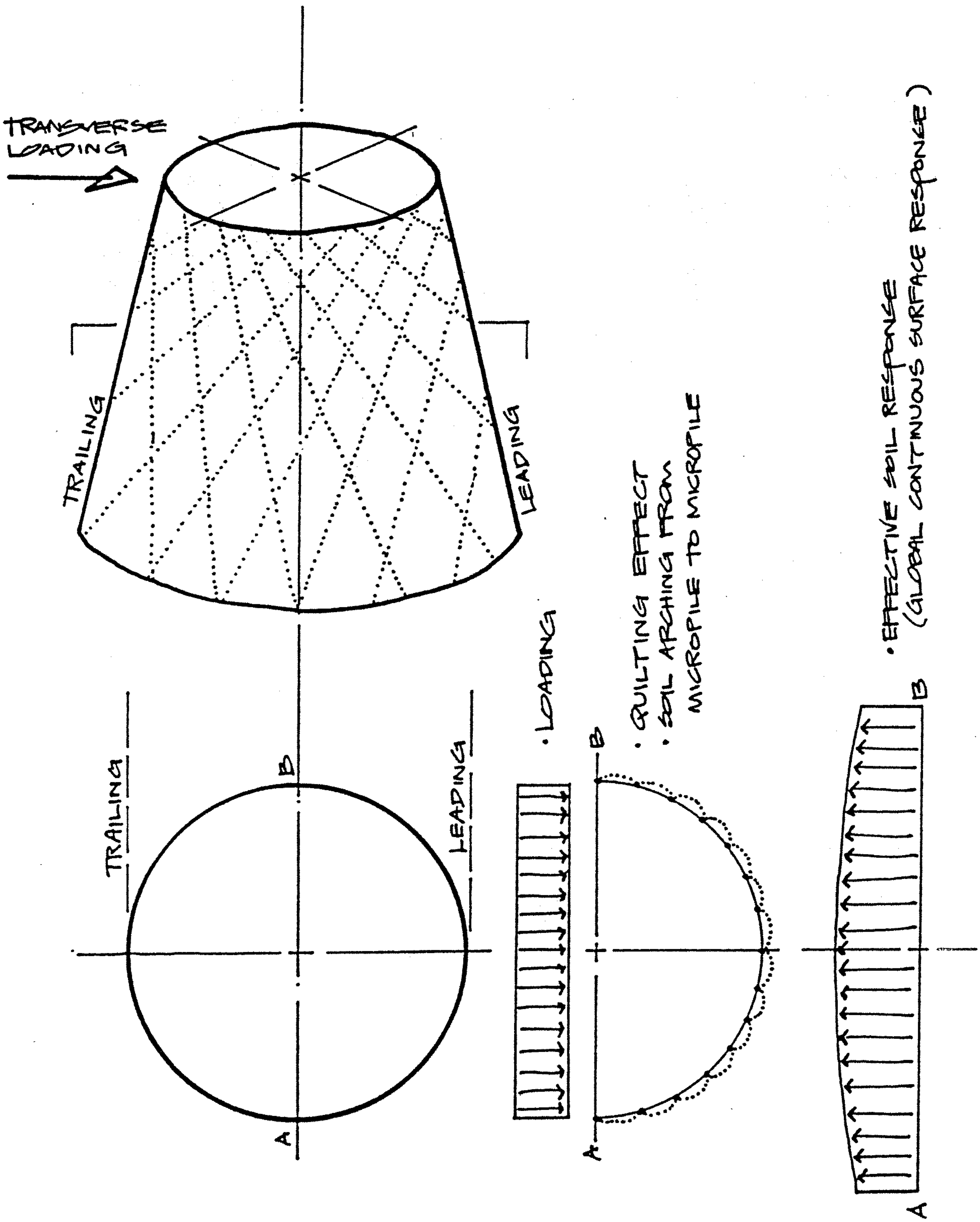


Figure 9. Global Continuous Surface Response From Quilting Effect

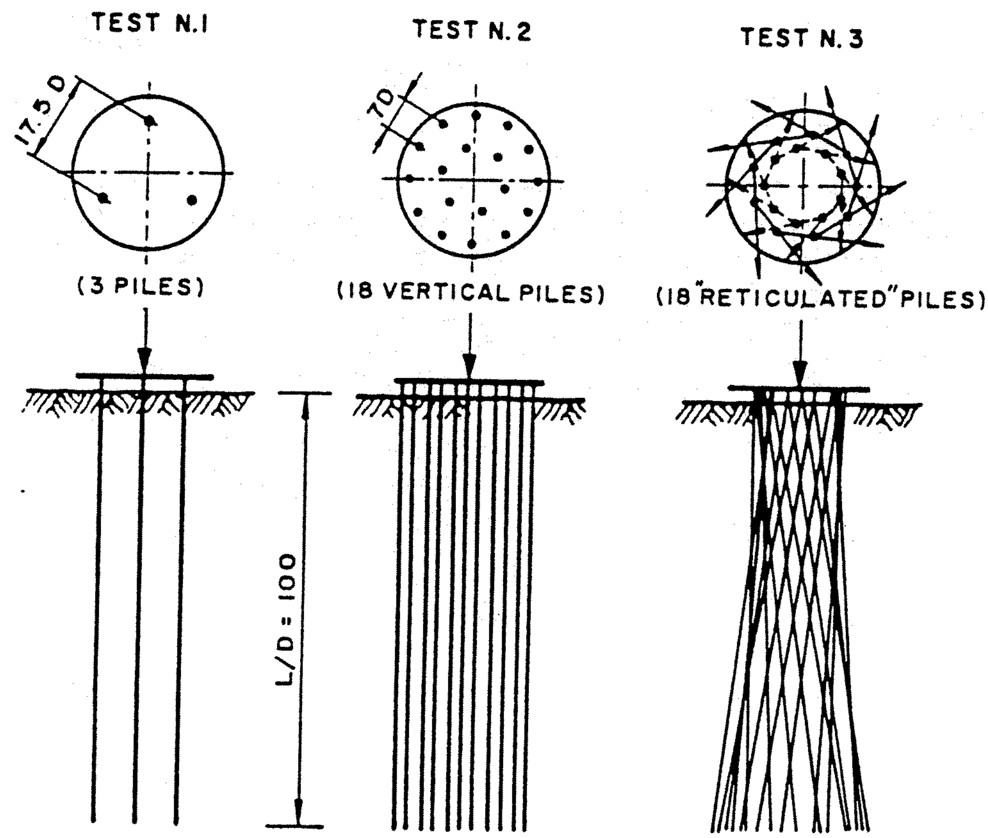


Figure 10. Arrangement of Piles in Lizzi's Original Model Tests

Source: Lizzi 1994

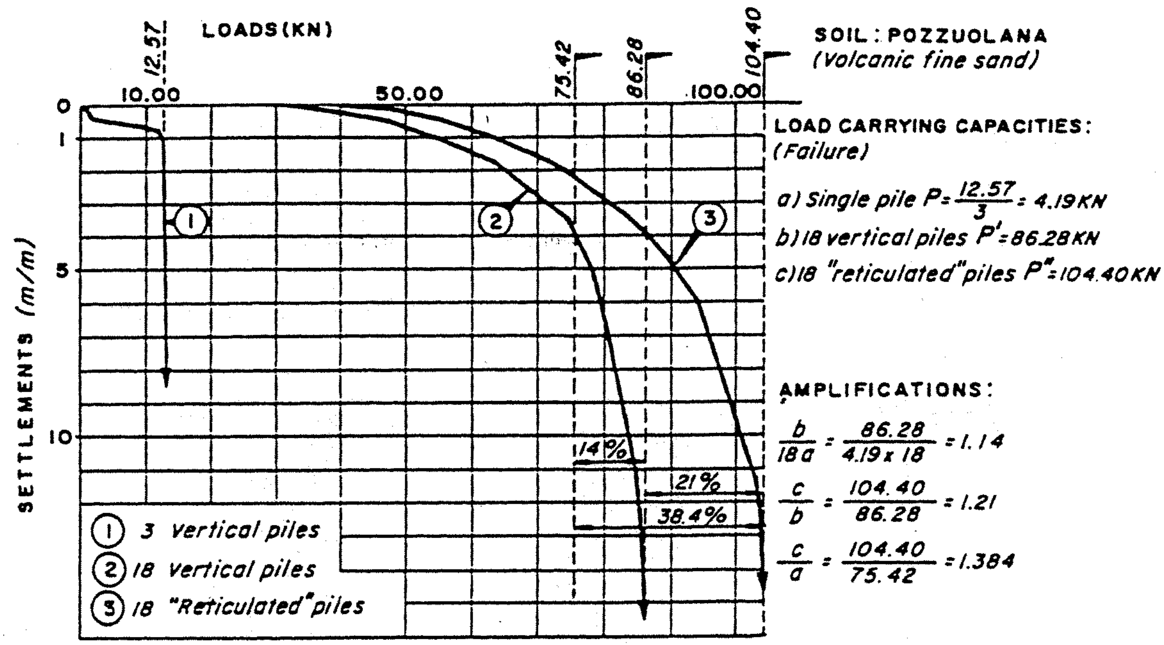


Figure 11. Load Test Results for Piles in Pozzulana (Fine Volcanic Sand)

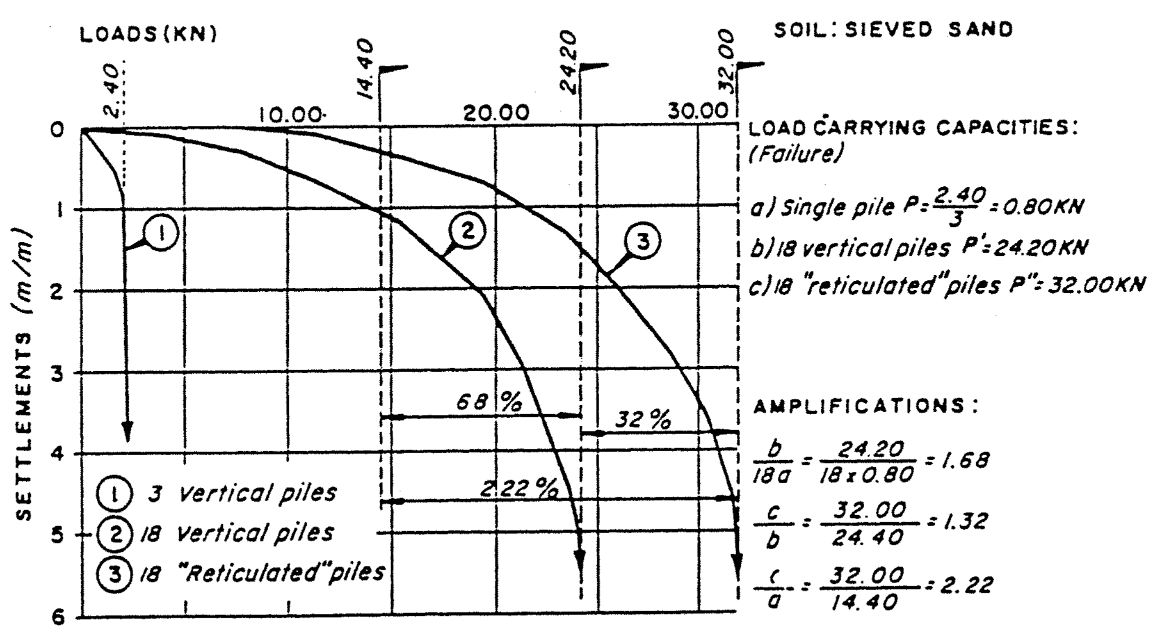


Figure 12. Load Test Results for Piles in Sieved Sands

Source: Lizzi 1994

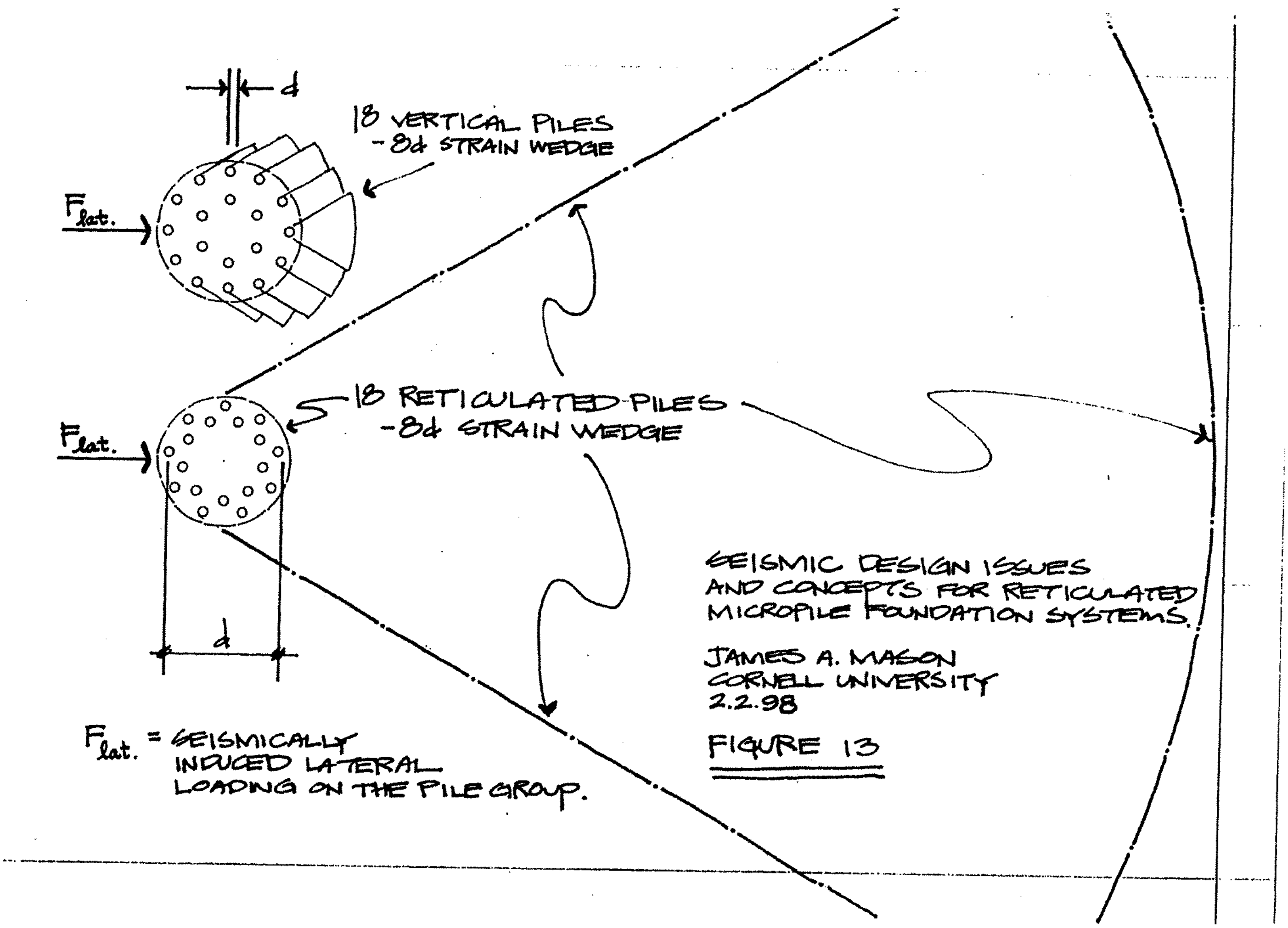

d
18 VERTICAL PILES
- 8d STRAIN WEDGE
$F_{lat.}$
18 RETICULATED PILES
- 8d STRAIN WEDGE
$F_{lat.}$
d
$F_{lat.}$ = SEISMICALLY INDUCED LATERAL LOADING ON THE PILE GROUP.
SEISMIC DESIGN ISSUES
AND CONCEPTS FOR RETICULATED
MICROPILE FOUNDATION SYSTEMS.
JAMES A. MASON
CORNELL UNIVERSITY
2.2.98

FIGURE 13