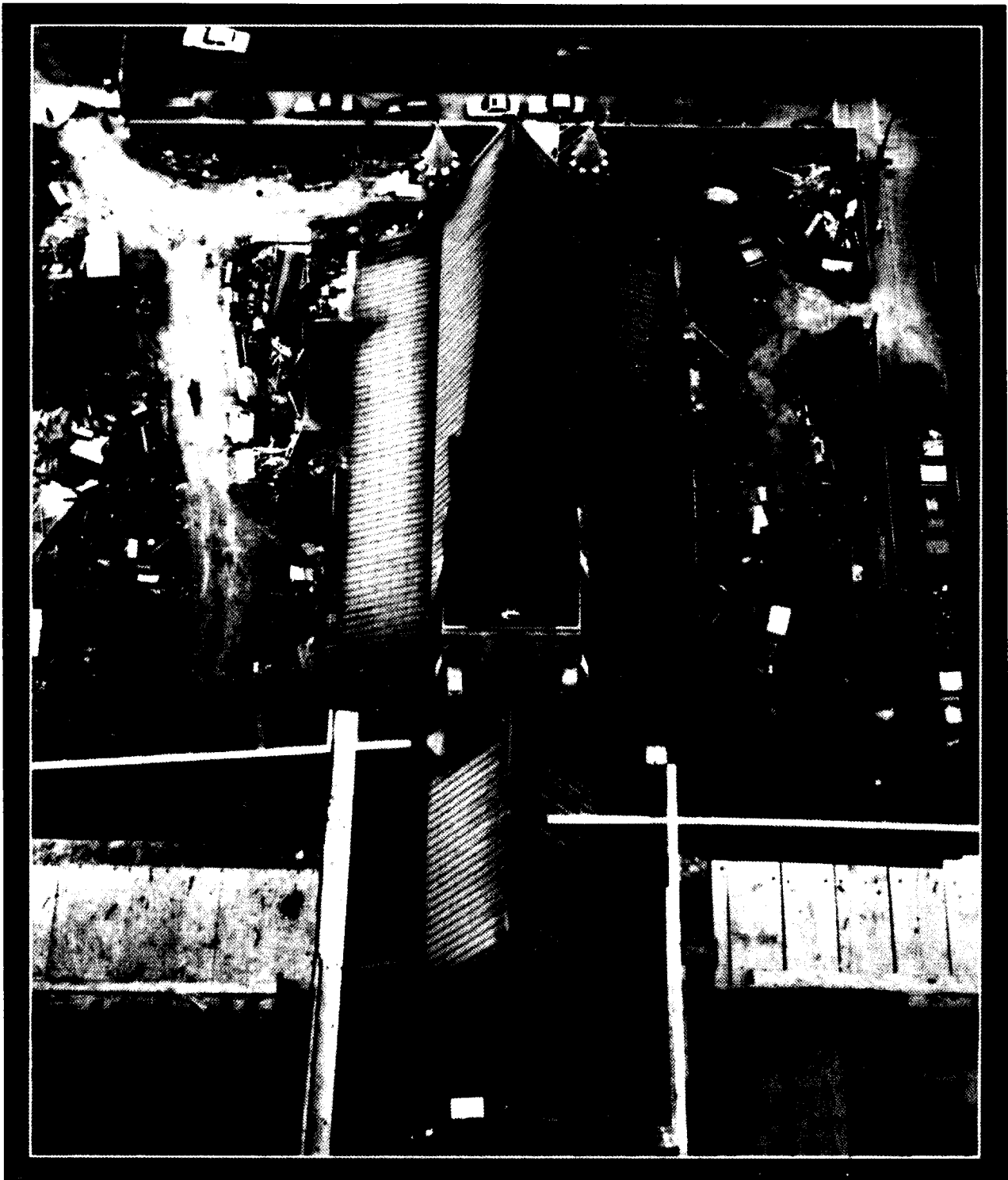


CONCRETE FOUNDATIONS



Compilation 19

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Contents

3	Subgrade Reaction in Mat Foundation Design, by Edward J. Ulrich, Jr.	38	Machine Foundations — A Case Study, by H. Marzouk
13	Underpinning Christ Church Cathedral, by Dirk J. Dressel, Joseph Gallaccio, and Shahbaz Mavaddat	41	Building Pile Caps on Weak Soils, by W. Tiner, E.J. Ulrich, and P.D. Moore
18	Foundation Design in Florida Karst, by John E. Garlanger	46	Concrete Foundations on Permafrost, by Alfred R. Mangus
25	Building a Big Mat, by Myer Dimentberg	49	Design of Foundations in High Settlement Areas, by Thomas J. Tepper
28	Post-Tensioned Residential Foundations, by Jack W. Graves, Jr.	58	Pumping for Slurry Walls, by Gilbert R. Tallard
32	Trench Wall Footings, by Purushottam Mehta	61	What Does a Soil Report Really Tell Us? by Robert L. Goehring
34	Massive Placement Melds Technology With Economy, by Emad Youssef	65	Casting a Large Concrete Slab in Hong Kong, by D.J. Sharpe, A.G. Ford, and M.G. Watson



Preface

ACI Compilations combine material previously published in Institute periodicals to provide compact and ready reference on specific topics. The material in a compilation does not necessarily represent the opinion of an ACI technical committee — only the opinions of the individual authors. However, the information presented here is considered to be a valuable resource for readers interested in the subject.

Hugh S. Lacy

Chairman, ACI Committee 336

Combined Footings and Pier Foundations

Erick N. Larson

Chairman, ACI Committee 351

Foundations for Equipment and Machinery

Jerry A. Holland

Chairman, ACI Committee 360

Design of Slabs on Grade



American Concrete Institute, Box 19150, Redford Station, Detroit, Michigan 48219

Subgrade Reaction in Mat Foundation Design

by Edward J. Ulrich, Jr.

A long time ago, before man became enlightened as to the scientific mysteries of the earth, the wise and foolish builders were identified: "... a wise man built his house on the rock. When the rains came down, the streams rose, and the winds blew . . . , yet it did not fall because it had its foundation on the rock . . . a foolish man built his house on sand. The rain came down, the streams rose, and the winds blew . . . and it fell with a great crash." (Matthew, chapter 7).

As man gained knowledge and expertise, wisdom seemed to degenerate, and man began to build where ever a need arose, often on sand or "soil." As a result, today we have the mat foundation and its relationship with the soil, a relationship as complex and unpredictable as that between a man and woman. Often an apparently compatible relationship between mat and soil deteriorates due to the ever-present plague of construction details and the design-construction relationship.¹

Subgrade reaction, the most important variable in designing mat foundations, is reviewed in four accompanying case studies of four landmark buildings in Houston. Broad concepts related to mat foundation analysis using the finite element method will acquaint the practitioner with the related salient soil-structure interaction concepts. A comprehensive examination of structural considerations in connection with mat foundation design is furnished by others.^{2,3}

Using an individual, uniform coefficient of subgrade reaction will allow analysis of mat foundations; however, a uniform modulus of subgrade reaction as the soil model

often is not rational and will mislead the designer.

Past structural practice

In the past, analyzing mat foundations included two major simplifying assumptions, an infinitely rigid mat and one-way bending.⁴ For very stiff mats with symmetrical geometry and loading, these assumptions may not introduce serious error. Assuming an infinitely rigid mat allows determination of a bearing pressure distribution by simple statics, which ignores the high pressure concentrations in the load application areas and the low pressure concentrations in areas distant from the load applications. The second assumption provides for the mat behavior to be approximated by a one-way, rather than a two-way, bending analysis.

Two analysis methods used to address the limitations and errors of these simplifying assumptions are the successive approximations method and the method of finite differences. These are extremely tedious and cumbersome procedures that do not lend themselves to an efficient general computer implementation. The successive approximations method addresses the mat flexibility relative to the bearing strata but does not include analyses for two-way bending. The method of finite differences appears to adequately address mat rigidity and two-way bending.

Finite element method

The finite element method (FEM) is easily implemented in a generalized computer code and eliminates the need for oversimplification. Some needed capabilities offered by FEM which provide a more realistic analytical model are:

- consideration of two-way bending
- comprehensive mat and bearing stratum interaction using the beam-on-elastic foundation concept
- unusual and complex mat shapes
- mats of significant thickness differences
- mats accepting large moments and axial forces from laterally loaded shearwalls or frames
- mats in which structure rigidity affects mat behavior and stress distribution

Although most engineers are aware of FEM, the method has not enjoyed widespread use. But due to the major advances in microcomputer technology of the mid-1980s, which have made vast computer capabilities available and affordable to most users, FEM will be used increasingly on most projects requiring mat foundations.

FEM analysis details and assumptions

An FEM analysis is based on the theory of flat-plate bending with the mat supported by the soil. The soil is modeled as a dense liquid using Winkler springs. The mat is modeled as a mesh of discrete elements interconnected at the node points, and the Winkler springs are used as the soil-response model at each node. Some basic assumptions made using FEM include:

- the mat acts as an anisotropic or, isotropic, homogeneous, elastic solid in equilibrium
- the subgrade reactions are vertical vectors and are proportional to the deflection of the node
- the subgrade reaction is equal to the spring constant at a node multiplied by the deflection of that node

Types of FEM solutions available for mat analysis along with the assumptions, advantages, and disadvantages are summarized in Reference 2. The most important variable in analyzing a mat foundation is the subgrade response.³ But even with the most superior soil mechanics theories and computer analyses, subtle construction details, the design-construction relationship, and low-quality construction can ruin the design.

FEM analysis results

Results of a mat analysis using FEM may be divided into three groups: deflections, bending moments, and subgrade reactions. The deflections are computed at each node based on the interaction of the loading mat-soil system. Three moments may be reported at each stress reporting point. These may consist of the bending moments per unit length, in a direction parallel and perpendicular to the x -axis, and the twisting or torsional moment.

The mat support reaction output presents the reactions at each of the boundary conditions defined for the analysis. These reactions may be used to determine applied contact pressures at the mat base by dividing the reaction force by the contributory surface area of the mat at each node. Some FEM analysis programs include a subroutine that performs the computation so that the support reaction is furnished as a contact pressure.

Subgrade response model

Theory of subgrade reaction

Unlike a simple supported beam, the mat foundation obtains its support from the earth. Although the earth may be in contact with the entire mat foundation, the response characteristics needed to analyze mat foundations properly are not available with the degree of precision that exists for simple beams with known material properties.

The theory of subgrade reaction⁵ is available to obtain approximate solutions in analyzing mat foundations. Winkler introduced the concept of subgrade reaction into applied mechanics in 1867, and the concept was expanded in subsequent years to include the computation of the stresses in such flexible foundations as continuous footings or mats. To get reasonably accurate results, the coefficients of subgrade reaction — spring constants — must be compatible with the deformation characteristics of the subgrade, loaded area characteristics, and the loading characteristics.

An individual, uniform coefficient of subgrade reaction has been used to analyze mat foundations, although using an individual value is not rational. The actual subgrade response from mat foundations leads to mat deflections and contact pressures that result in nonuniform coefficients of subgrade reaction. Differences in a uniform coefficient of subgrade reaction may have little effect on computed bending moments, but varying values of the coefficient of subgrade reaction across a mat foundation area can have a major impact on computed bending moments and reinforcing steel requirements.

The theory of subgrade reaction is based on two simplifying assumptions:^{5,6}

- the subgrade obeys Hooke's law; the ratio k between the contact pressure p and the corresponding displacement y is independent of the pressure
- the subgrade reaction on the base of a rigid, centrally loaded plate resting on the horizontal surface of the subgrade has the same value at every point of the base; the coefficient of vertical subgrade reaction k has the same value for every point on the surface acted upon by the contact pressure

The coefficient of subgrade reaction k_s is the ratio between this pressure at any given point on the surface of contact and the subgrade deformation y produced by the load application at that point. The coefficient of subgrade reaction may be expressed by the equation

$$k_s = p/y$$

The fundamental principles of the theory are illustrated by Fig. 1, but the illustration indicates that the mat deflection is equal to the total subgrade movement. The vertical subgrade movement that occurs as the mat deflects may often be only part of the total subgrade deflection during the operating life of a structure. The theoretical value of the coefficient of subgrade reaction is affected by a number of factors⁵ including foundation size, shape, depth, and subsurface conditions, and character of load application.

Bearing capacity

The foundation pressures must first be compatible with the subsurface bearing capacity to apply the theory of subgrade reaction in analyzing a mat foundation. Constant ratios exist between stresses and strains according to elastic theory. But for elastic theory to have some degree of applicability in analyzing a mat foundation, the real requirement is not that the subsurface material necessarily be elastic, but that constant ratios must exist between stresses and corresponding strains.⁷

General, local, and punching shear considerations should be examined in the bearing capacity analysis. Local bearing capacity considerations may not be significant for mat foundations in deep excavations where the excavation wall is at the mat edge, such as in the Enron Building. General, local, and punching shear considerations may be very critical on projects where the stratigraphy incorporates major differences in strength, or on projects where the excavation wall does not extend to the mat edge, such as in Texas Commerce Tower, First Republic Bank Center, and Transco Tower. Only three garage basement levels were permitted for Texas Commerce Tower to satisfy bearing capacity requirements.

The compatibility of the foundation pressures and bearing capacity is usually examined by applying a factor of safety to the ultimate bearing capacity to obtain allowa-

uation of differential movement between columns on the mat and columns supported by foundations outside of the mat; and, (3) assistance in designing architectural finishes which, from the writer's limited experience, has often provided the most severe foundation design criteria for buildings.

Traditional soil response prediction to compute mat deflections

In association with the mat analysis using a selected coefficient of subgrade reaction, mat settlement was computed for short-term and sustained loading conditions by assuming the net soil pressure is distributed uniformly over the base of the mat. The settlement values were then compared with mat deflection values computed with the constant coefficient of subgrade reaction and adjustments were made in the mat stiffness and subgrade reaction coefficients to obtain mat deflections that were compatible to the settlement predictions or the structural engineer's interpretation of the settlement predictions.

The long-term subgrade response characteristics make using a constant coefficient of subgrade reaction too simplifying an assumption. The coefficient of subgrade reaction is not constant for a mat foundation and may vary considerably across the mat. Small differences in coefficients of subgrade reaction across a mat may result in only small differences in deflection, but the computed bending movements can vary by 20 or 30 percent for deflection differences of a few tenths of an inch.

Discrete area method

The need to provide a more rational analysis of subgrade response led to the discrete area method,^{9,11} which allows the geotechnical engineer to properly model the subgrade response for each system of mat contact pressures determined by the selected loading condition and to furnish coefficients of subgrade reaction for each system of contact pressures in the analysis.

After general, local, and punching shear bearing capacity analyses indicate an acceptable bearing pressure and the mat thickness is proportioned to satisfy the punching shear provisions of the ACI Build-

ing Code, the mat analysis continues using the finite element method. The analysis follows an interactive procedure in which the computed mat deflections are compared to the subgrade deflection computed by the geotechnical engineer (Fig. 2) for the loading conditions and loads that will be imposed upon the mat.

A range of loading conditions may be analyzed with this method.

The discrete area method procedures used in analyzing mat foundations follow.

1. For the loading condition that is to be analyzed, the geotechnical engineer produces a single coefficient of subgrade reaction, modi-

ENRON Building

The ENRON Building is a 52-story, 1,440,000-ft² (133,920-m²) structure. The tower mat is located at a depth varying from 36.75 to 46.50 ft (10.9 to 14.19 m) below street level (Fig. 2). The structural analysis and design process are well documented.¹⁶

Foundation system — The mat foundation approximates the plan shape of the tower and has an overall length of approximately 300 ft (91.4 m) and a width varying from 150 to 154 ft (45.7 to 47 m) (Fig. 1). The mat extends about 25 ft beyond the exterior columns. Mat thickness varies from a basic dimension of 8.5 ft (2.59 m) to 19.25 ft (5.87 m) at the service elevator pit areas. The mat foundation, cast in what is reported to be the largest single mat placement ever, contains 13,308 yd³ (10,180 m³) of concrete. Careful attention was given to monitoring/limiting the concrete temperature during the

placement and to making sufficient provisions with steel reinforcement for resisting shrinkage/temperature cracking.¹⁵

An intricate array of rebar trusses were installed below all column locations having significant wind uplift forces. Very careful detailing was used to minimize local flexing of the mat. The goal of forcing the mat to move and tip as a monolithic element when subjected to wind loads was accomplished.¹⁶

Predicted and observed mat deflections — Gross computed mat contact pressures at the final iteration using the discrete area method are given in Fig. 2. Predicted and observed mat deflections are presented in Fig. 3. Mat deflections were computed to range from about 6 in. (15 cm) at the building interior to nearly 4 in. (10 cm) along the exterior column line. The observed mat deflections are substantially less than those predicted (Fig. 3).

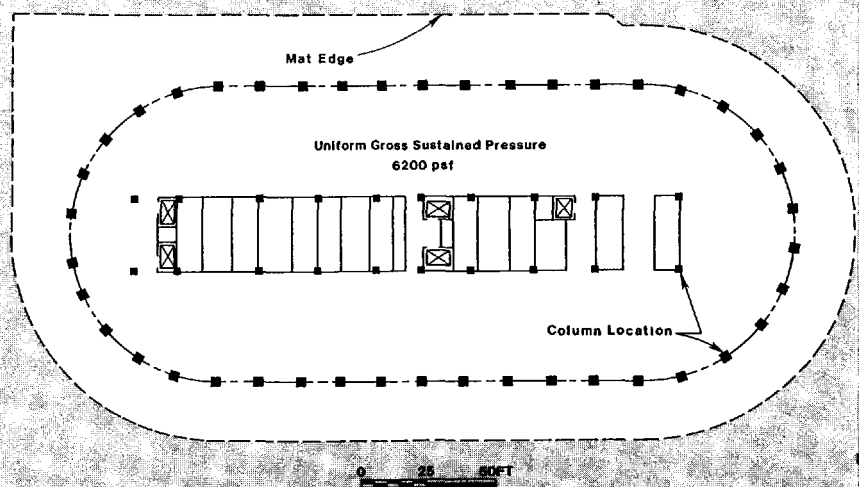


Fig. 1 — ENRON Building foundation plan with column locations.

fied according to Terzaghi⁵ and Teng⁶ and adjusted by judgment, to allow the first analysis of the mat using the finite element method.

2. The output, consisting of deflections and contact pressures is then idealized into small discrete areas of equal contact pressures by the geotechnical and structural en-

gineers. Some finite element programs include subroutines that convert loads at the nodes to pressures and then generate contours of equal contact pressure. Small discrete areas are preferred over large areas.

3. After the mat is idealized into small discrete areas, the geotechnical engineer computes the subgrade

response for the idealized system of loaded areas.

4. The subgrade response is compared to the mat deflections for compatibility. If compatibility does not exist, new coefficients of subgrade reaction are computed based on the contact pressures and new individual soil responses.

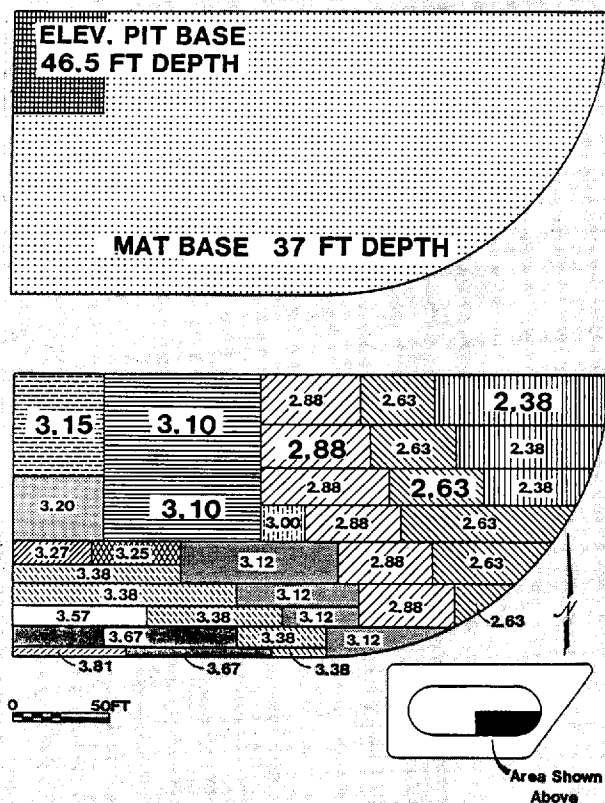


Fig. 2 — ENRON Building contact pressure distribution at final iteration using discrete area method and sustained loads.

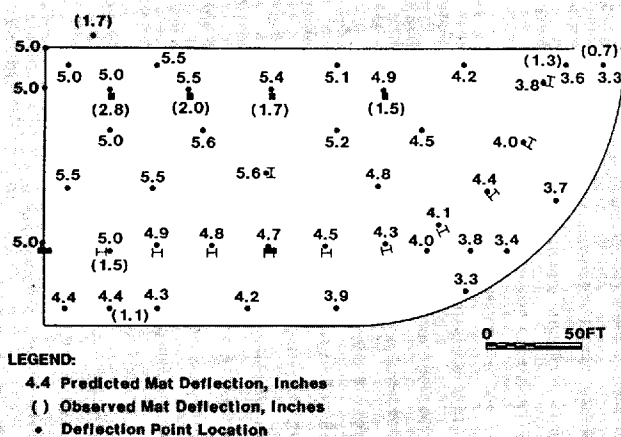


Fig. 3 — ENRON Building predicted and observed mat deflections for sustained loads.

Transco Tower

Located in the essentially suburban complex around the Galleria, 10 miles (16 km) west of the Houston Central Business District, Transco Tower is the tallest building outside of a central business district in the United States. At 64 stories, the building is 901 ft (275 m) high and has multiple setbacks at 15 ft (4.6 m) intervals. The structure is approximately square in plan and has a steel frame with a exterior tubular design. The exterior columns are placed at 15 ft (4.6 m) on center. The corner columns were excluded so that the spandrel beams run on the diagonal to reduce shear-lag. A parking garage connects to the tower's west side above and below ground by bridge and a vehicle service tunnel.

Foundation system — The building rests upon an 8 ft (2.4 m) thick mat approximately 200 ft (70 m) square with a basement that extends beyond the mat (Fig. 1). The bottom of the mat is 37 ft (11.3 m) below the surrounding ground surface. Design wind loads are believed to create an overturning moment of about 3×10^6 kip-ft (414,674 Mg-m).

Predicted and observed mat deflections — The Transco Tower foundation plan with column locations on the mat and basement wall is given in Fig. 1. Areas of computed equal gross sustained contact pressures developed in the final iteration of the mat analysis using the discrete area method are shown in Fig. 2. Predicted and observed mat deflections are presented in Fig. 3. The predicted mat deflections suggest the mat should dish downward at the center and tilt toward the east. The observed mat deflec-

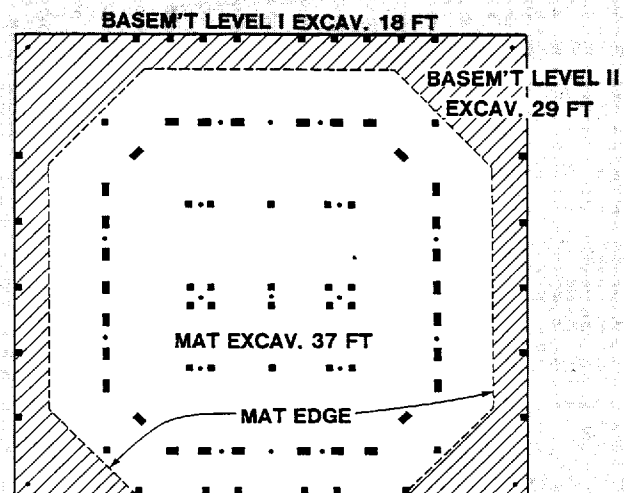


Fig. 1 — Foundation plan with column locations.

5. The mat analysis is again performed using the new coefficients of subgrade reaction. New contact pressures and deflections are given and the mat is again divided into areas of equal contact pressures. The geometry of these new areas may be different from those used in the first iteration.

6. The subgrade response is again computed and compared for compatibility with the mat deflections.

7. The procedure continues until deflection compatibility is achieved.

Deflection compatibility may be achieved by three iterations. Although at least two loading condi-

tions should be analyzed — total (fixed load + live load + wind load) and sustained (fixed load + sustained live load), the critical loading conditions will depend on the project. Analyses and performance of building foundations^{8,9,11,17} have shown that the more severe loading conditions may be from

tions indicate mat deflections similar to those predicted, but the observed mat deflections have only been part of the total observed mat settlement.

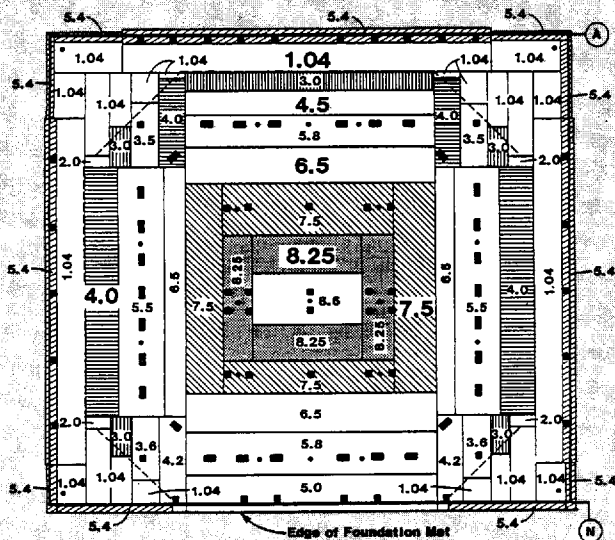


Fig. 2 — Transco Tower contact pressure distribution at final iteration using discrete area method and sustained loads.

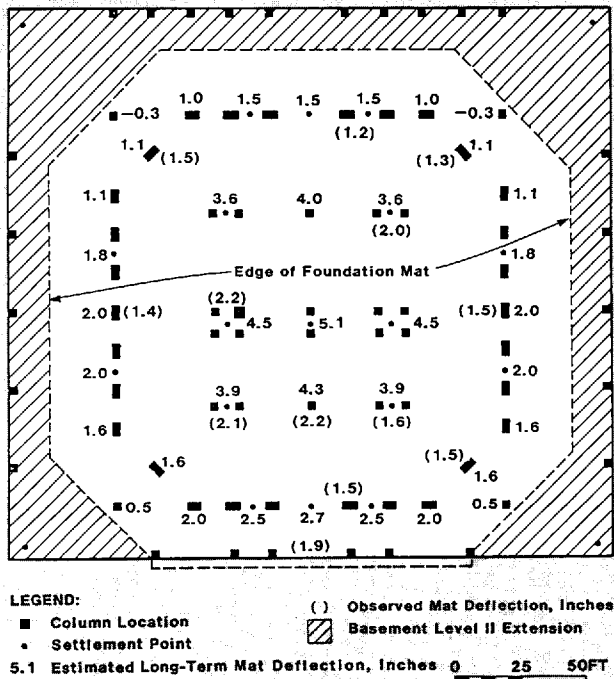


Fig. 3 — Transco Tower predicted and observed mat deflections for sustained loads.

Texas Commerce Tower

Texas Commerce Tower is Houston's tallest building at 75 stories and was the world's tallest soil-supported building when completed in 1981. The building still is believed to be the tallest composite building in the world.^{14,17}

Shown in profile and plan in Fig. 1, the building consists of a tower with a lightly loaded attached parking garage and covers a full block in the Houston Central Business District, about 260 ft² (79 m²). There are four basement levels within the tower area and three levels in the garage outside the tower. Although the tower's lowest floor level extended to about 53 ft (16.2 m) deep, the parking garage level was limited to 41 ft (12.5 m). The building measures 160 ft² (48.8 m²) in plan, with one corner having a large chamfer at a 45-deg angle to create a fifth side

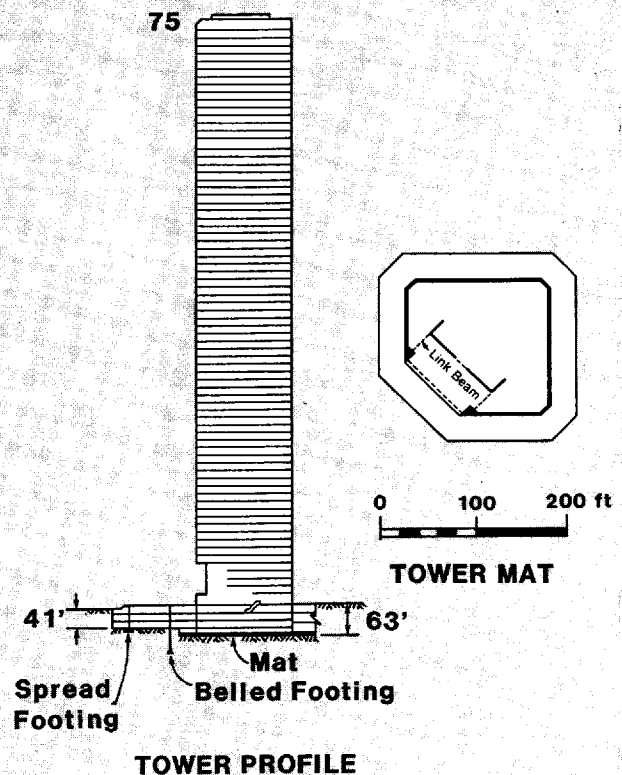


Fig. 1 — Profile and mat plan of 75-story Texas Commerce Tower.¹⁵

fixed and sustained loads instead of from the total applied loads. Coefficients of subgrade reaction computed using the discrete area method may differ by a factor of ten, depending on the subsurface characteristics and loading conditions. This range will significantly influence the mat design.

The subgrade response analysis techniques may be based on elastic theory, elastic-plastic theory (consolidation), FEM, or a combination of each theory. The soil response computation selected will depend on the loading conditions to be analyzed, subsurface conditions, and local experience of the geotechnical

engineer. Some available computer programs for subgrade response have methods of computing stress increases due to a rectangular, uniformly distributed loaded area at or below the ground surface using the distribution theories of Boussinesq, Mindlin, Westergaard and below grade (Westergaard).^{7,12,13}

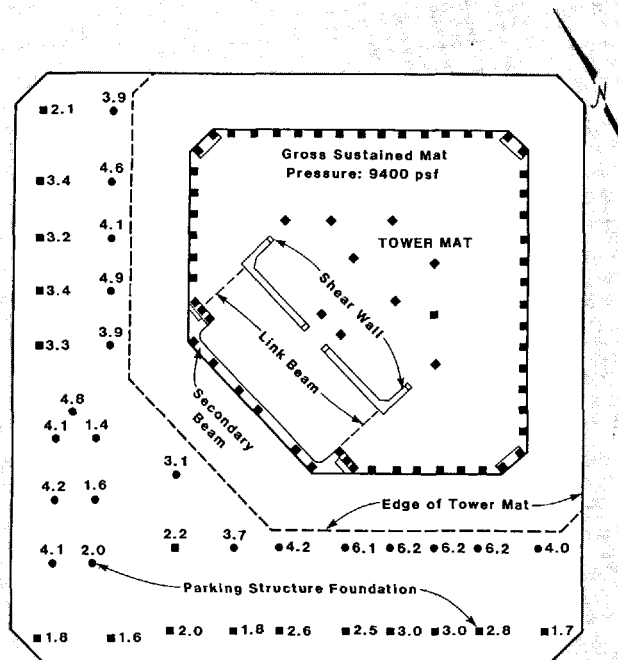
that is an 84-ft (25.6-m) column-free span of steel girders and dual-pane glass.

Foundation system — The tower is founded on a concrete mat 9 ft-9 in. (3 m) thick that extends 27 ft-6 in. (8.4 m) beyond the exterior columns (Fig. 2). The bottom of the mat is 63 ft (19.2 m) below ground floor level, making the mat excavation the deepest in Houston's history at the time of construction. Dewatering was required to provide a dry excavation and for the operating life of the building. A detailed presentation of the structural system details and features is given by Banavalkar¹⁴ and Ulrich.¹⁵

The sustained foundation pressures used in predicting mat deflection are given in Fig. 2. A uniform gross pressure of 9420 psf (450 kPa) was used to compute mat deflections for sustained loading con-

ditions. The influences of individual foundations supporting the garage were considered using the pressures shown in Fig. 2.

Predicted and observed mat deflections — Predicted and observed mat deflections are presented in Fig. 3. Predicted mat deflections ranged from about 3.5 in. (9 cm) along the exterior columns to almost 5 in. (13 cm) near the shearwalls. The observed mat deflections related to angular rotation between the columns at the northwest corner and shearwall is computed to be about 1/150. Observations indicated the mat deflections were about 2 in. (5 cm) near the exterior columns (Fig. 3), but close to 5 in. (13 cm) near the shearwalls. Observed mat deflections amounted to only part of the settlement under long-term loading conditions.



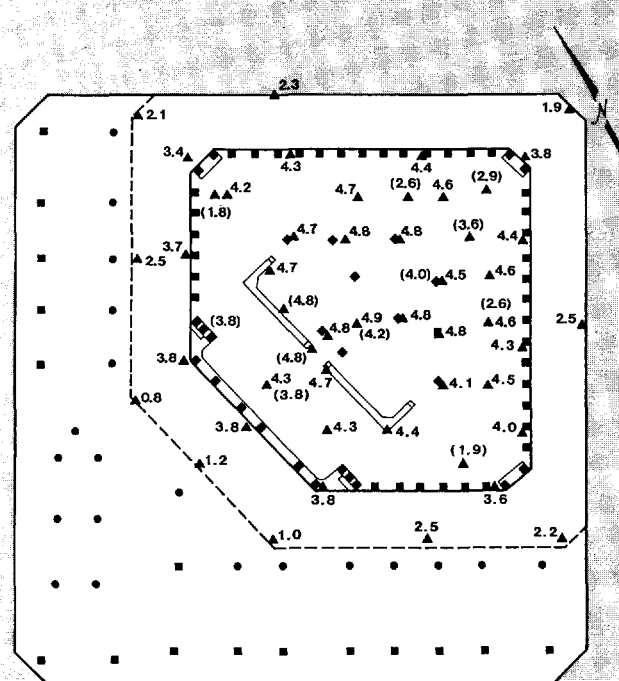
LEGEND - GARAGE:

- Column Location with Spread Foundation
- Column with Drilled-and-Underreamed Foundation

NOTE:

Numbers at column locations indicate gross sustained foundation pressure in ksf.

Fig. 2 — Texas Commerce Tower sustained foundation pressures.



LEGEND:

- Column Location
- Column with Drilled-and-Underreamed Foundation
- ▲ Stress Point Location Beneath Mat
- ▲ Predicted Long-Term Mat Deflection, Inches
- () Observed Long-Term Mat Deflection, Inches

Fig. 3 — Texas Commerce Tower predicted and observed mat deflections for sustained loads.

Limitations

Predictions of subgrade response are estimates at best. Mat analysis is subject to significant limitations in spite of the advances in computer technology of the 1980s. Though not often admitted, the degree of confidence in predicting load transmitted to the mat decreases dramatically as the structure becomes larger and more complex. The subgrade response model and parameters are not precise and are an idealization of a material that has inherent variability that challenges the most sophisticated models.

The methods used to analyze the subgrade response model require the results of well instrumented performance studies of major structures and nearly flawless knowledge of subsurface conditions to predict the subgrade response with a high degree of confidence. The natural point-to-point variability of the subsurface conditions, though a cliché, is real and magnifies the uncertainty of the subgrade response model. The design-construction relationship and construction details are probably the most important limitation to an accurate mat analysis.

Poor construction methods can ruin the results of the most sophisticated analyses.

Case studies

The accompanying case studies of four Houston landmark buildings offer additional insight into the subgrade response part of the mat analysis. The discrete area method was used in each building foundation design except the 75-story Texas Commerce Tower. Although short- and long-term loading conditions were considered in each mat analysis, only the subgrade analysis

First RepublicBank Center

First RepublicBank Center occupies a Houston Central Business District block. The development consists of two major structures: a 12-story atrium banking hall 125 ft (38 m) tall on the east half of the city block, part of which envelops the existing Western Union communications center, and a 56-story, 777-ft (252-m) stair-stepped office tower occupying the remaining half block (Fig. 1). Both the banking hall and tower have basements that extend four stories — 48 ft (14.6 m) below street level and 38 ft (11.6 m) below the Western Union building foundations. The design and construction concepts including the existing Western Union building are shown in Fig. 1. Structural features of First RepublicBank Center are reported in Reference 9. Generalized to-

tal applied loads to each mat segment are given in Fig. 2.

Foundation system — The tower is supported by an 8 ft thick (2.4 m) concrete mat extending about 55 ft (16.8 m) below street level. The banking hall rests upon spread footings at about 50 ft (15.2 m) depth in the parking garage, and a line of 48 in. (121.9 cm) diameter drilled piers which form the north basement wall at the Western Union building; one bank hall column rests on the northeast corner of the mat, and one column rests upon a pair of belled piers with

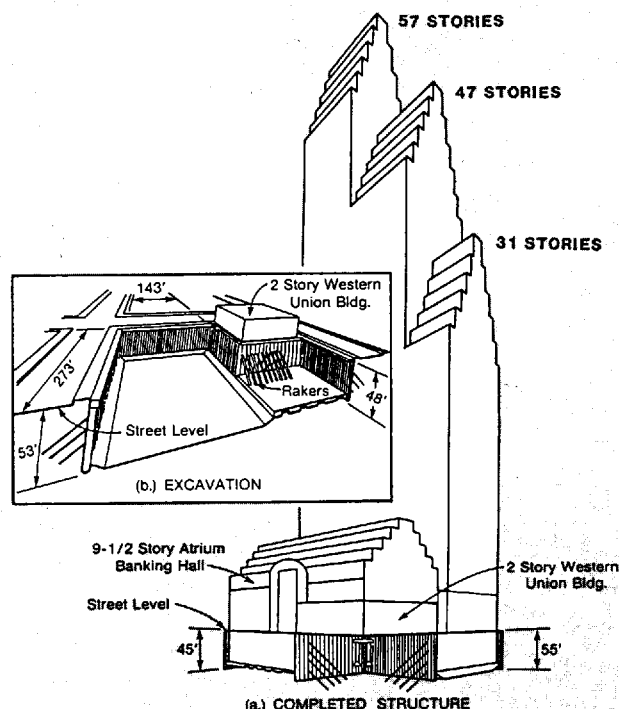


Fig. 1 — First RepublicBank center views.

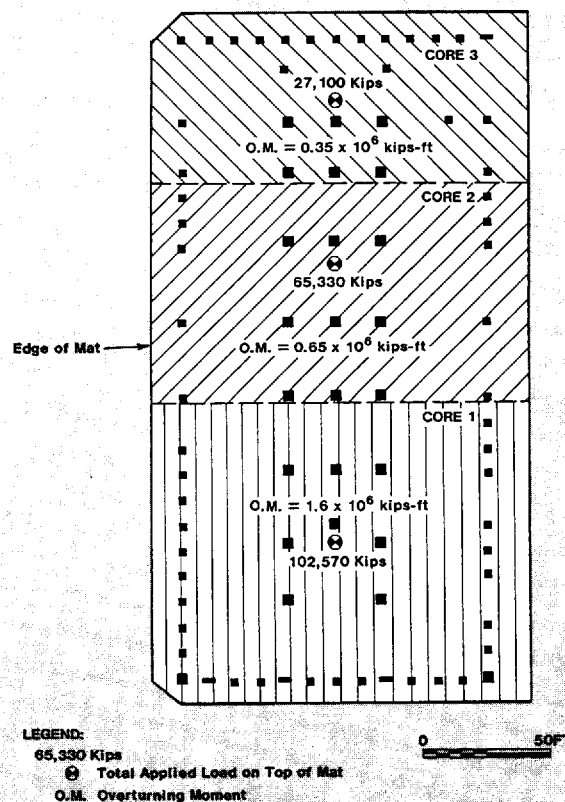


Fig. 2 — First RepublicBank Center mat: total applied load generalized to top of mat segments.

results for the long-term loading conditions are presented because the long-term loads were found to impose the more severe design requirements after determining the mat thickness. Subsurface conditions at the sites are overconsolidated clays and sands having characteristics similar to those given in Fig. 3.

Conclusions

Using a uniform coefficient of subgrade reaction to analyze and design mat foundations is an over-

simplification of the soil response and will lead too wrong designs. The coefficient of subgrade reaction may vary considerably across a mat. After analysis of bearing capacity, the discrete area method should be used to model mat-soil interaction. Accurate predictions of the subgrade response remain the fundamental, but elusive, element of the mat analysis.

Innovative design to reduce costs and coupled with increased risks are mandated to the design team; but in spite of the leaps in computer technology the subgrade response pre-

diction is an estimate at best for large mat foundations with complicated loading conditions. Although the discrete area method is available to allow the mat analysis to properly model subgrade response for the effects of complicated mat loading on the subgrade, the subgrade model and parameters contain imperfections that only can be improved through more rigorous construction observations with comprehensive measurements of the mat and subgrade behavior.

The mat foundation relationship with the subgrade is complex. Con-

13 ft (4 m) diameter bells. The northeast corner of the Western Union building was eliminated, and structural elements supporting walls, floors, and roofs were permanently shored with an independent foundation system.

Predicted and observed mat deflection — The gross mat contact pressures for the sustained loading condition developed in the final iteration using the discrete area method are given in Fig. 3. The building weight is less than the weight of soil removed. Contours of pressure decrease are shown in

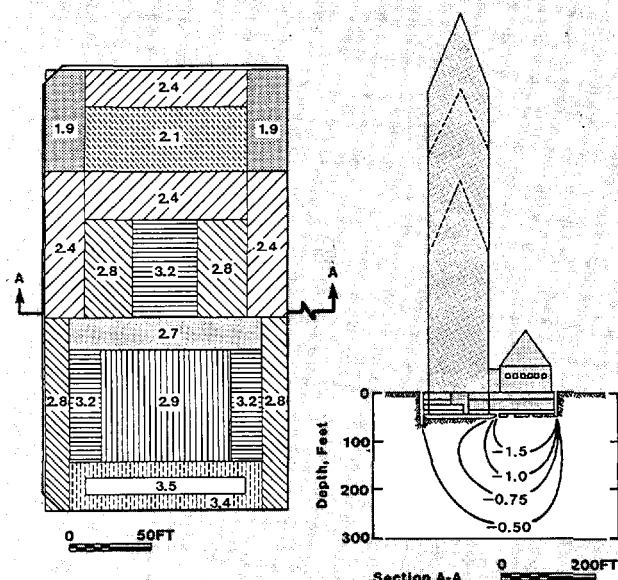


Fig. 3 — First RepublicBank Center contact pressure distribution at final iteration using discrete area method and sustained loads.

Fig. 3 to present an interpretation of the stress changes on the soil for a section that included the bank hall.

Values of predicted and observed mat deflections are given in Fig. 4, along with profiles of the predicted deflections. The values of movement shown for the bank hall foundations are presented only to offer an example of the influence the lightly loaded structure may have on the predicted mat deflections.

Observed mat deflections were less than those predicted. Predicted mat deflections ranged from a few tenths of an inch (2 to 5 mm) to over 2 in. (5 cm), depending on the location and calculated pressure distribution.

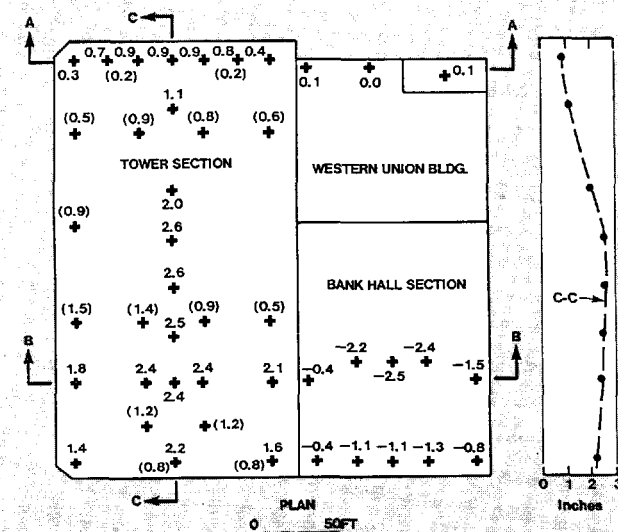


Fig. 4 — First RepublicBank predicted and observed mat deflections for sustained loads.

struction details and the design-construction relationship offer perhaps the most severe limitations on perfecting the subgrade response model. Comprehensive systems of observations and measurements are needed by the design team during below-ground construction to evaluate the as-designed foundations and to make adjustments. Inadequate construction quality control and inadequate observations with unsatisfactory measurements continue to cause unsuitable mat performance.

We must become wise builders if we are to "build the house upon the sand."

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ACI member **Edward J. Ulrich, Jr.**, is principal engineer of Ulrich Engineers, Inc., Houston, Texas. He has over 20 years experience in geotechnical engineering related to industrial, power, and commercial facilities and has directed the geotechnical services for some of the world's tallest soil-supported buildings. He is past chairman of ACI Committee 336, Combined Footings and Pier Foundations, and a member of Committee 543, Concrete Piles.



Underpinning Christ Church Cathedral

by Dirk J. Dressel, Joseph Gallaccio, and Shahbaz Mavaddat

Intense commercial development in downtown Montreal, where prime land is scarce, resulted in a unique arrangement between private developers Firstcliff and the Diocese of Montreal, owners of Christ Church Cathedral. The Cathedral is a Victorian Gothic structure built in 1859 of load-bearing stone masonry, which, together

with its surrounding lot, occupies an entire city block between two major shopping centers.

The commercial development scheme called for a 34-story office tower on the lot behind the church and for a three-level underground retail mall and an underground parking garage directly below the church. The main condition, how-

ever, was that the Cathedral remain fully intact, undamaged, and operational throughout the construction period, and that all construction underneath the church be confined to the existing 4 ft (1.2 m) to 7 ft (2.1 m) deep basement.

It is this condition that gave rise to the challenging problem of fully supporting the historical Cathedral

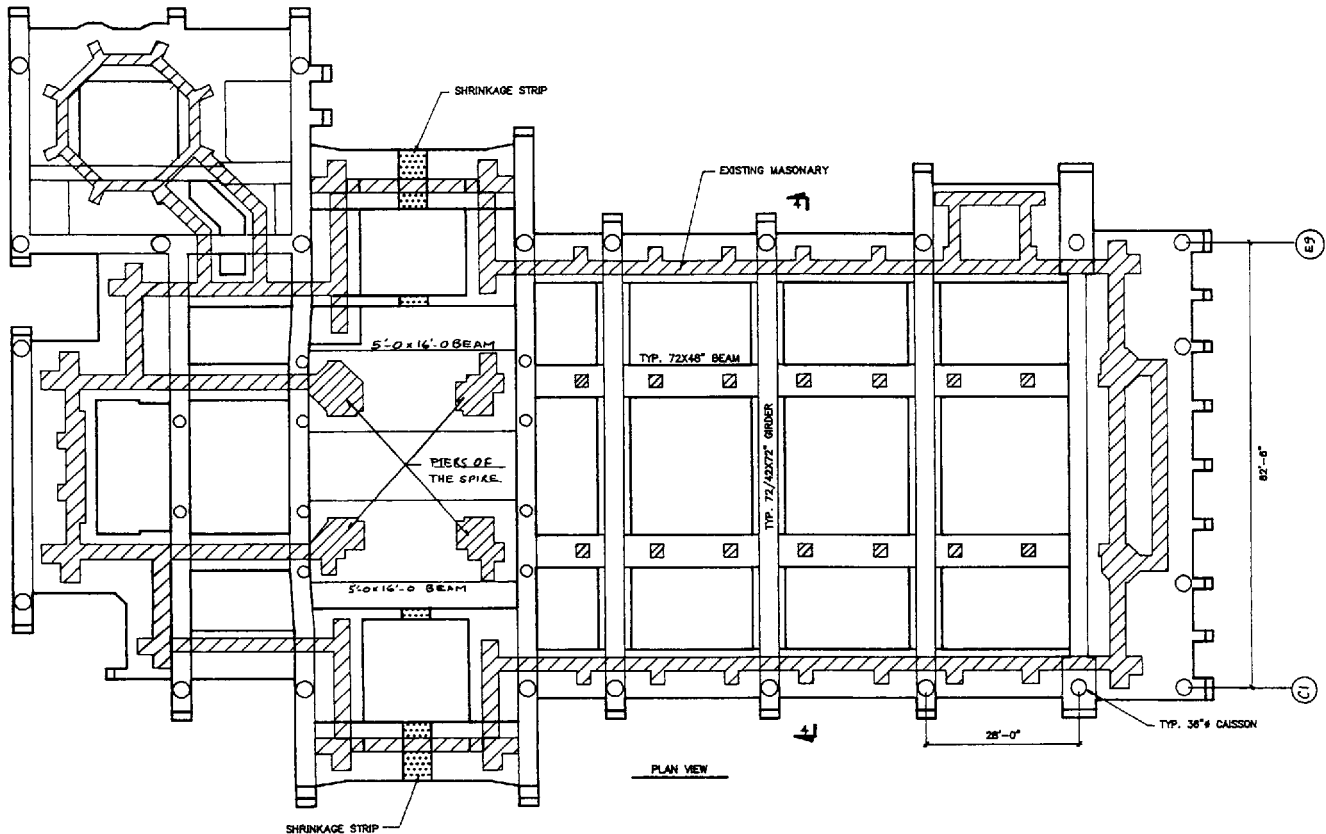


Fig. 1 — Plan of existing Cathedral and the new underpinning concrete structure.



Fig. 2 — Installing caissons outside the perimeter walls.

on new foundations to allow excavation of four stories beneath it.

Existing conditions

As shown on Fig. 1, the existing Cathedral structure is spread over some 16,000 ft² (1500 m²). Perimeter load-bearing masonry walls and interior columns along the main longitudinal aisle are the main vertical supporting elements, founded on masonry spread footings below the existing floor of the church's basement. These footings were supported on clay underlain by glacial till, with bedrock at approximately 45 ft (13.5 m) below ground.

The central masonry tower with its metal spire rises 220 ft (66 m) above the existing timber floor of the church. The tower was founded on inverted stone arches that were underpinned in 1939 because of settlement problems.

Phase I — the church

The main constraints in dealing with providing a new foundation structure for the church were:

- The church would have to remain fully operational, precluding removal of the existing timber floor to allow easy access into the basement, and therefore limiting the work to the existing basement with its restricted headroom.
- The scheme would have to maximize the column-free space for the retail area under the church, resulting in long spans of 82.5 ft (24.7 m) over the width of the church.
- The existing 18,000-ton (16,345-Mg) masonry structure above grade would have to remain undamaged, with no alterations or modifications, and the design would have to achieve zero deflections.
- The new retail area under the church would require a three-hour fire separation from the existing timber floor of the church.

Faced with these problems, the structural engineers, Quinn Dressel Associates, devised a cast-in-place post-tensioned concrete scheme that could provide an economic solution within the given constraints. A grid of post-tensioned beams and girders would grip the existing masonry structural members at the basement level and transfer the loads to new concrete-filled steel caissons on the perimeter of the church walls. The phases of design and construction have been covered in more detail in Reference 1.

Fig. 1 shows the plan of the post-tensioned grid for the front part of

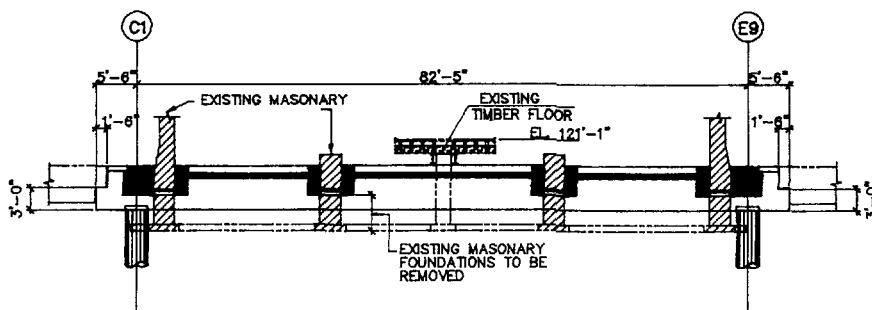


Fig. 3(a) — Section of new concrete grid supporting existing masonry.

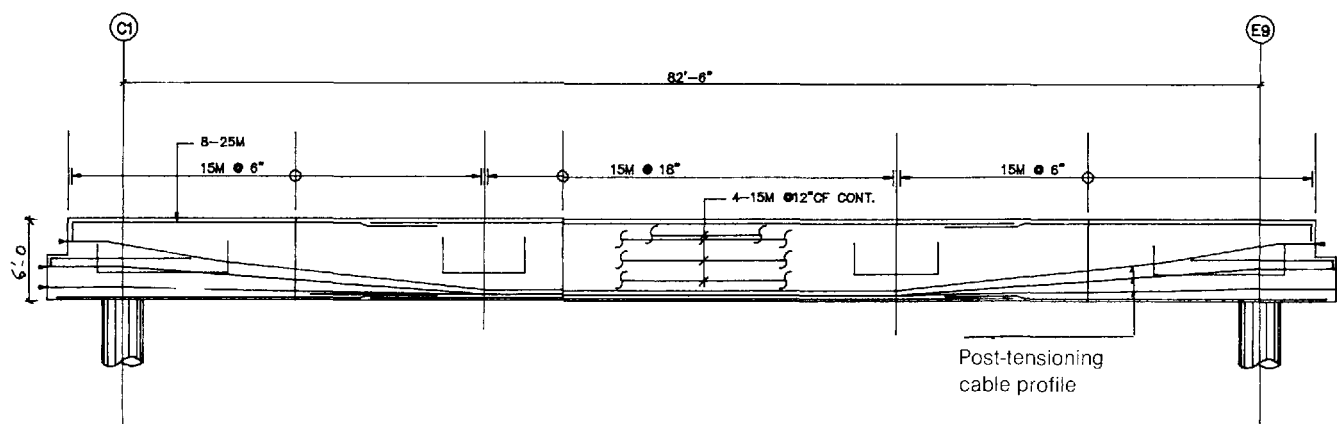


Fig. 3(b) — Typical girder elevation.

the cathedral. The underlying design principle for the new supporting structure is best illustrated by considering a typical girder and beam on Fig. 1. Steel caissons, 36 in. (900 mm) in diameter, were installed outside the church walls (Fig. 2), socketed into bedrock, and then filled with concrete. These would support 6 ft (1.8 m) deep girders spanning 82.5 ft (24.7 m) across the width of the cathedral (Fig. 3).

A secondary system of 4 ft (1.2 m) deep beams was designed to enclose all existing masonry foundation elements (Fig. 4) and transfer their load to the main girders. Secondary beams were transversely post-tensioned with single bar tendons to ensure friction transfer of loads from masonry foundation elements to the secondary beams. The secondary beams were then post-tensioned longitudinally to transfer the loads to the main girders. Since

they were relatively short in length, about 13 ft (3.9 m), concentric post-tensioning cables were provided to ensure adequate friction transfer of loads and strength. Deflections were anticipated to be very insignificant as the beams provided ample stiffness.

Finally, the main girders were post-tensioned to transfer the loads to the perimeter caissons. The cable profile (Fig. 3) in the girders was such that the applied dead weight could be counterbalanced by the post-tensioning forces, ensuring zero deflections. Several analyses were performed to trace the load path on the grid and to study the interaction between the enclosed masonry elements and the surrounding concrete.

Two extreme cases were considered in the design: one was to consider the masonry integral with the surrounding concrete, the other was

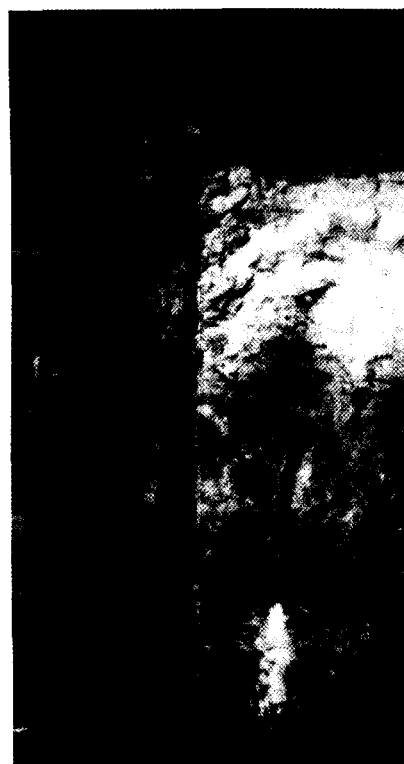


Fig. 4 — Existing interior basement masonry, with drilled holes for post-tensioning tendons.

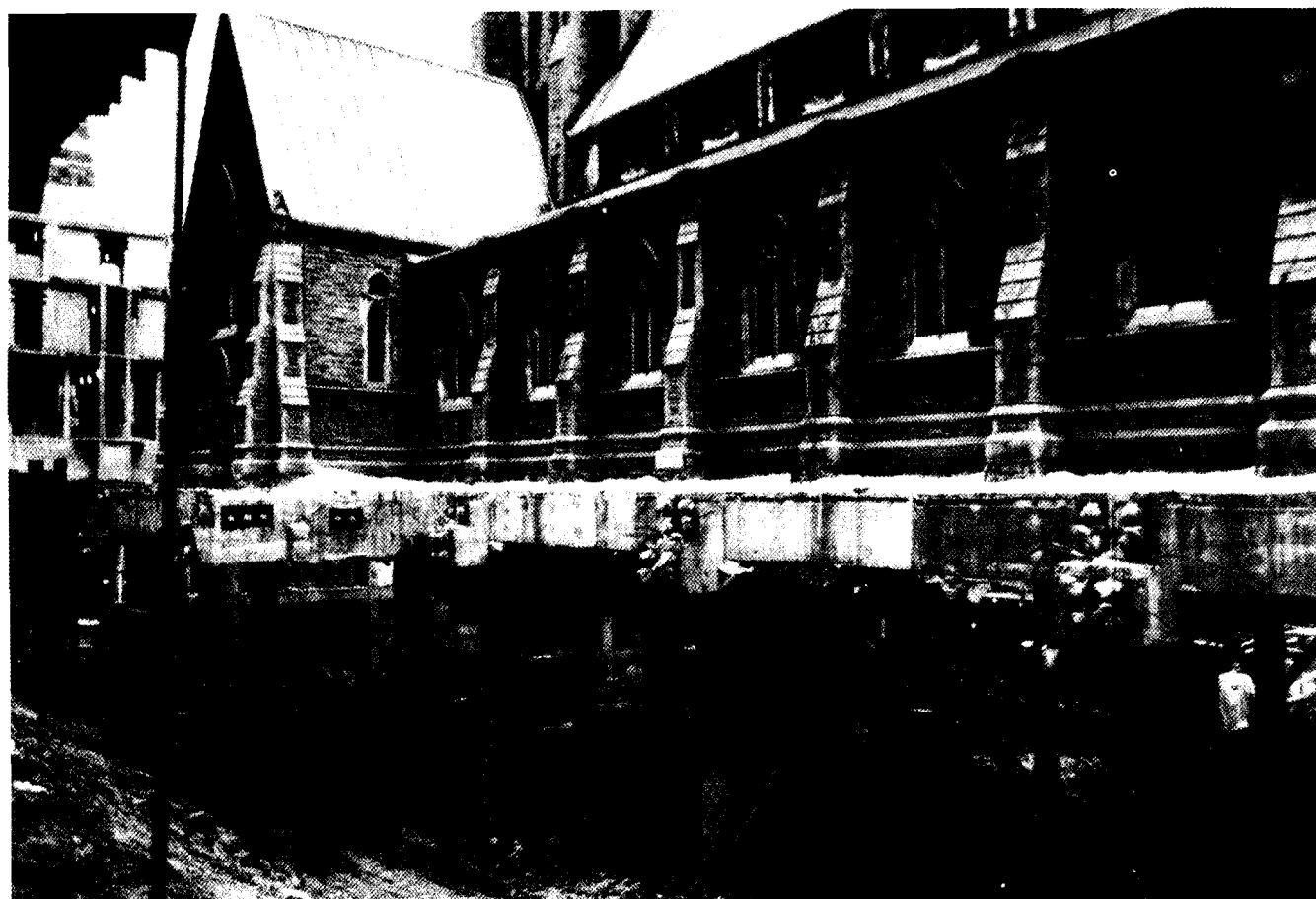


Fig. 5 — Completed excavation.



Fig. 6 — Excavation under the existing spire foundation, exposing 5 ft of the existing piles.



Fig. 7 — New concrete transfer beams supporting the spire foundations.

to assume an opening in the concrete where it was interrupted by masonry. In either case, concrete stresses at critical sections remained within allowable limits for service loads. Grid analysis and two-dimensional beam analysis were performed to accurately predict deflections that could then be eliminated by suitable post-tensioning. These

analyses showed insignificant differences in the load path. However, in areas where the supporting structure consisted of a wide strip, and where it was not readily apparent whether the strip would act as a slab or as a beam, several analyses were performed, each having a different assumption with respect to the distribution of loads.

Although results indicated no significant difference for various assumed load paths, provisions were made for applying additional post-tensioning in case it became necessary after the loads were transferred. But after making these provisions in one instance, the practice was abandoned. As well, grouting of the post-tensioning ducts in all other areas was delayed so that some of the post-tensioning forces could be released in case any cambering occurred. Deflections were monitored at all stages of construction and these precautions were not required. The maximum vertical movement was measured at about $\frac{1}{4}$ in. (5 mm).

After all post-tensioning was complete for this phase, excavation began below the church. Temporary steel bracing was installed between the caissons and the main girders to ensure lateral stability for the supporting structure, which would stand 50 ft (15 m) high once the excavation was complete (Fig. 5). The bracing was removed once retail floors were built.

Phase II — the spire

The existing 3000-ton spire was originally supported on four large stone piers, joined together by inverted masonry arches. By the 1920s, settlement had caused the tower to lean 4 ft (1.2 m) to the south. Remedial measures at the time included jacking and underpinning the stone foundations with concrete-filled steel pipe piles. The new post-tensioned supporting structure would have to be placed at the underside of the original stone piers. A 1:1 soil berm was maintained between the excavation in Phase I and the working space under the spire.

The concrete transfer beams, 16 ft (4.8 m) wide by 5 ft (1.5 m) deep, were designed to span 40 ft (12 m) between 26 in. (650 mm) diameter caissons. The beams were then post-tensioned to pick up the full load from the spire and its masonry foundations. To make room for machines to drive the caissons to rock and to pour the new beams, it was necessary to excavate under the

existing stone piers, exposing the piles from the earlier underpinning.

The piles were assumed to be carrying their load by a combination of skin friction and end bearing. To compensate for the portion of skin friction lost by the 5 ft (1.5 m) of excavation (Fig. 6), concrete brackets were designed to transmit a portion of the spire load to two post-tensioned main girders in the immediate vicinity of the spire. Once the pipe piles were exposed, the existing inverted arches were removed and shear studs were welded to the pipes to provide for load transfer to the new beams (Fig. 7).

Long-term effects of post-tensioning and movement were all considered in the design. The new structure was divided into two parts with a shrinkage control strip. Specifications for concrete did not have to be anymore stringent than usual.

Despite the complicated nature of the work, no major difficulties were

encountered and the post-tensioned concrete scheme proved to be quite versatile and adaptable to the existing site conditions. No interruptions were caused to church services during the eight-month construction period and the church structure suffered no damage other than a broken window during the caisson-driving operations. Furthermore, the new underpinning system stabilized the existing foundations, which had a long history of settlement.

The flexibility of the technique became quite apparent in providing for a realistic definition and control of the load paths and for permitting the accurate prediction and virtual elimination of deflections.

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Dirk J. Dressel is the principal of Quinn Dressel Associates, Consulting Structural Engineers, Montreal, Canada. He has over 30 years experience and has been the recipient of several awards, including one for this project and one for the Scotia Plaza high-rise in Toronto. ACI members **Joseph Gallaccio** and **Shahbaz Mavaddat** are senior engineers with Quinn Dressel Associates. Their experience includes analysis and design of many challenging building projects.



Dressel



Gallaccio



Mavaddat

Foundation Design in Florida Karst

by John E. Garlanger

The entire Florida peninsula is underlain by solution-weathered limestone, with cavities in some areas that are known to exceed 100 ft (30 m) in height and width. The lengths of these natural conduits are measured in miles. The sinkhole-dotted surface of the limestone is typically buried beneath significant thicknesses of overlying sediment, and the foundation hazards associated with building on the limestone generally are not visible from ground level.

Fig. 1 is a generalized cross section through a hypothetical site in the sinkhole-prone area of central Florida. The limestone is overlain by consolidated clays of Miocene age and unconsolidated sands of Pleistocene age. The groundwater table in the sand is typically within 5 to 10 ft (1.5 to 3 m) of the ground surface and the piezometric level in the underlying limestone is typically 40 to 80 ft (12 to 24 m) below ground surface. Of primary importance from a foundation engineering perspective is the sand-filled breach in the clay layer. Groundwater from the surficial aquifer flows through this breach and recharges the much more productive limestone aquifer.

From a hydrological perspective, this ability to recharge the lower aquifer, which is the principal source of potable, agricultural, and industrial water in central Florida, with water from the rain-recharged surficial aquifer is quite beneficial. However, as the water supply de-

mand in the deeper aquifer causes increases in the hydraulic head difference between the two aquifers, the potential for "piping" sand through the breach in the clay layer into an underlying cavity increases.

Piping occurs when a subterranean conduit or tunnel is eroded backward from a location where groundwater is discharging from an unconsolidated soil deposit, such as at a spring.¹ Once erosion begins, it proceeds backward along the line of maximum hydraulic gradient toward the source of seepage. The end result can be catastrophic for a building foundation.

In Fig. 2, sufficient sand has piped into the cavity system in the limestone to create a cavity in the overburden sands. When the cavity enlarges to a size at which the beam action of the overlying soil can no longer support itself, the roof collapses, resulting in a sinkhole at the ground surface (Fig. 3). Fig. 4 illustrates this phenomenon at a site in the Orlando metropolitan area.

In Fig. 5, the groundwater table is higher, the soil above the water table is cohesionless, and the available void space in the limestone is small. In this case, the depression at ground surface is more saucer-shaped and the subsidence is more gradual. However, because of the resulting settlement, a building foundation constructed above the depression would still have failed. Fig. 6 illustrates this phenomenon at a site near Brooksville, Florida.

If the cavity system in the limestone is large enough or the flow in

the limestone is great enough to carry the eroded material away, the size of the sinkhole is limited only by the thickness and stable slope angle of the overburden (Fig. 7). Fig. 8 illustrates this phenomenon at a site in Winter Park, Florida. The sinkhole shown is over 350 ft (107 m) in diameter and is the largest sinkhole to have occurred in Florida during recorded history.

Site investigations in sinkhole-prone areas

The first step in designing a building foundation in central Florida is to locate the building away from the influence of potential sinkholes. This requires investigating a proposed building site to locate any breaches in the confining layer.

Typically, investigating sinkhole potential begins with studying the regional geology and hydrogeology and mapping historical sinkholes that have occurred in the project vicinity.

Fig. 9, which shows the occurrence of historic sinkholes in Polk County, Florida, was prepared in the early 1970s using data obtained from the Polk County Civil Defense Agency, conversations with local geologists, interviews of private individuals, and a search of newspaper articles. Since 1983, the Florida Sinkhole Research Institute at the University of Central Florida has been accumulating data on historic sinkholes for the entire state. This computerized data base has made it much easier to obtain information on historic sinkholes in

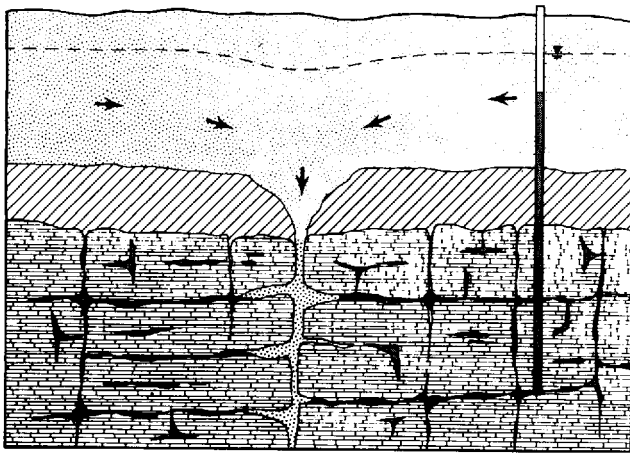


Fig. 1 — Generalized stratigraphic profile for sinkhole-prone areas in central Florida.

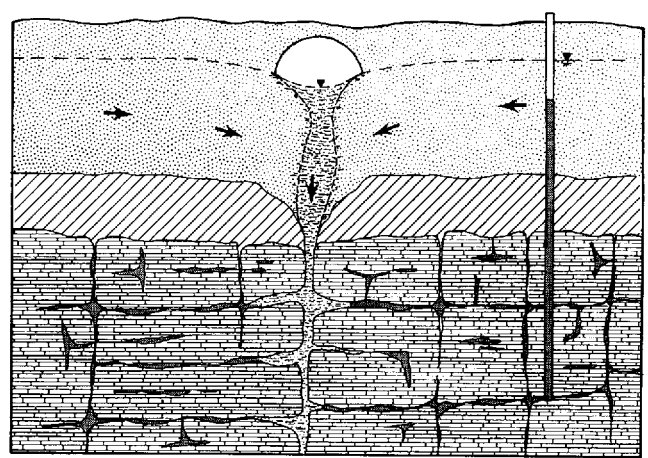


Fig. 2 — Early stage in the development of a cover collapse sink.

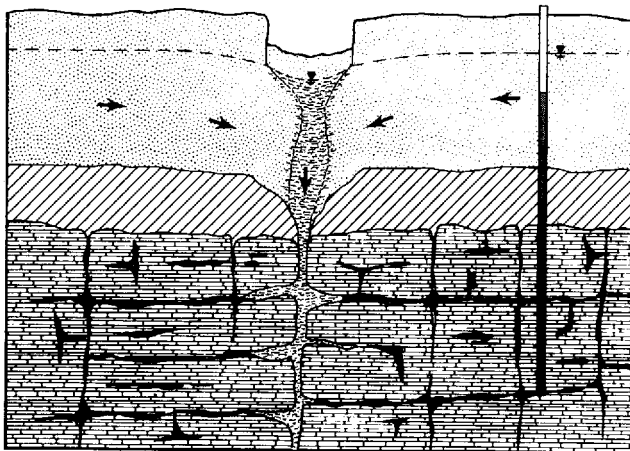


Fig. 3 — Final stage, cover collapse sink development.

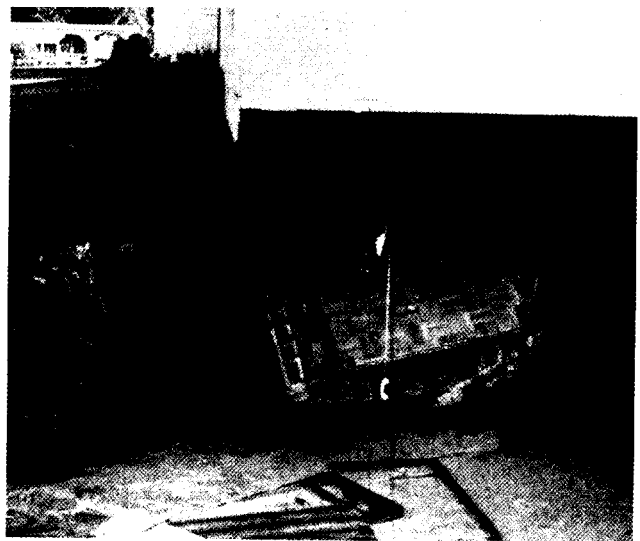


Fig. 4 — Cover collapse sink in metropolitan Orlando.

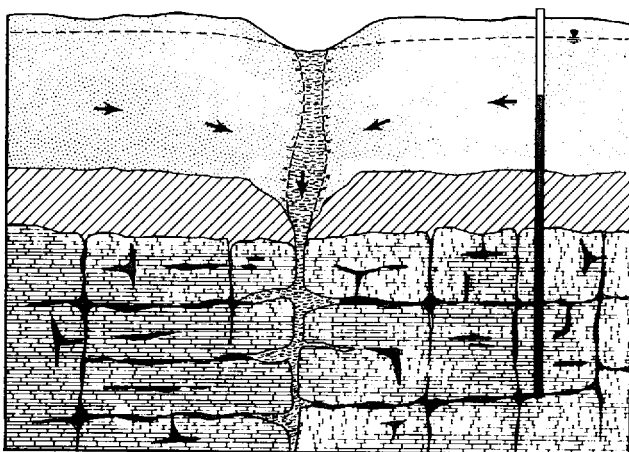


Fig. 5 — Development of subsidence sink.



Fig. 6 — Overburden subsidence sink near Brooksville.

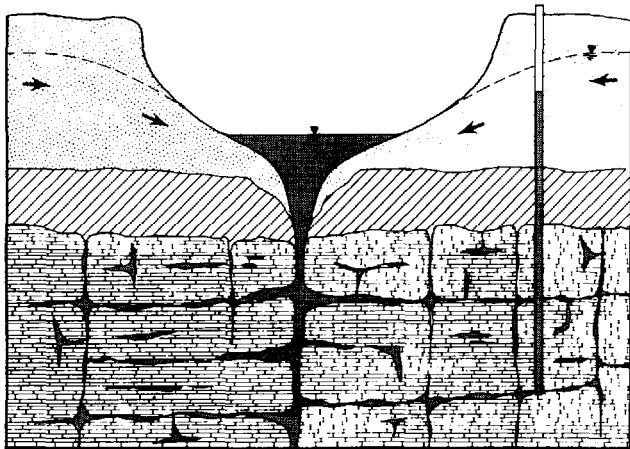


Fig. 7 — Development of ultimate overburden subsidence sink.



Fig. 8 — Overburden subsidence sinkhole, Winter Park.

the vicinity of a specific project site.

Fig. 10,² which was developed using the computerized data base, presents a moving average analysis of modern sinkholes for a 790-square-mile (1270 km²) region in Hillsborough County, Florida. The moving averages were based on a 4-square-mile (6.4 km²) quadrant with a 0.1-mile (0.16-km) step. This map indicates a highly localized distribution of modern sinkholes. The number of sinkholes ranged from 0 to 5 per square mile (1.6 km²) for the period of record. This corresponds to a frequency of sinkhole occurrence ranging from 0 to 0.2 per square mile per year.

Data from historic sinkhole mapping can also be used to locate areas on a proposed building site where a breach in the confining layer is more likely to exist. Because sinkholes in the limestone surface are most likely to have occurred at the intersection of two joints, it is possible to determine the potential location of breaches in the confining layer by mapping the joint systems.

Unfortunately, the joint system in the limestone typically is buried under overlying soil deposits; consequently, the joint system can only be inferred from such linear surface features as stream segments, alignment of lakes, and ponds, etc. These linear or near linear surface features are referred to as lineaments. Fig. 11 is a partial lineament map of south Orlando. Note that

there are two sets of lineaments, one which strikes N47°E and one which strikes N43°W.

Fig. 12 shows a proposed building site and the approximate location of all of the historic and prehistoric sinkholes that are known to have occurred in the immediate site vicinity. Also shown in Fig. 12 are bands representing the inferred location of joints in the limestone surface. These bands were constructed using the strike of the lineaments presented in Fig. 11. All of the site area crossed by the bands and particularly those areas containing the intersection of two bands is suspected of containing a breach in the confining layer.

If the building can be positioned to avoid all of the suspect areas and is outside the zone of influence of a potential sinkhole, the foundation exploration and design proceeds normally. However, if the building must be located near or over a suspected breach, a much more extensive subsurface exploration and testing program is required.

In many cases it is possible to begin the subsurface investigation using such geophysical techniques as ground-penetrating radar, EM conductivity, or seismic surveys.³⁻⁷ All of these techniques can detect anomalies in the subsurface profile. Fig. 13 is an example of a suspected breach located using ground-penetrating radar.

After all of the geophysical data has been reviewed and anomalous

areas located on the site map, a drilling and testing program is planned for the site. Within the areas of the site not located above a potential breach in the confining layer or paleosink in the limestone surface, the test borings or soundings are spaced conventionally. However, at least one of the test holes in this area is advanced into bedrock, i.e., into the confining layer, so that the unaffected site stratigraphy can be documented.

Within the suspect area of the site, the test holes are spaced much closer together, e.g., at each column location, and all of the holes are advanced to the top of the bedrock complex. Although Standard Penetration Test borings are still drilled on almost every major building site in central Florida, the cone penetrometer is becoming the sounding method of choice for most sinkhole investigations.^{8,9} The advantages of the cone penetrometer are speed and sensitivity to subtle changes in soil density. The electric piezocone is particularly attractive for sinkhole investigations because it can detect and document slight downward hydraulic gradients.

Fig. 14 illustrates a pore pressure profile within a sand-filled breach area where the downward hydraulic gradient is 1.3 ft/ft (0.4 m/m). Essentially all of the hydraulic head is dissipated in downward flow through the breach. Accurate water table measurements are also important on sites being investigated for

