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FOUNDATION FAILURES AND REPAIR: HIGH-RISE AND HEAVY CONSTRUCTION

SECTION 8

FOUNDATION FAILURES AND REPAIR: HIGH-RISE AND HEAVY CONSTRUCTION

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8.1 INTRODUCTION

8.1.1 High-Rise and Heavy Construction

Foundation failures of high-rise and heavy construction generally do not differ from those of low-rise structures and single residences. Foundation failures are all caused by human errors and are the result of either one or a combination of these three types of errors:

1. Errors of knowledge (ignorance)
2. Errors of performance (carelessness and negligence)
3. Errors of intent (greed)

This section will help to minimize future failures resulting from the first two types of errors. This famous quote says it well: "A wise man learns from the mistakes of others. Nobody lives long enough to make them all himself."

The consequences of high-rise failures are often very serious and most devastating because of the large scale and size of such structures. It is also self-evident that the potential for high loss of life is also great.

The pattern of foundation failures in high-rise structures is unique and differs from patterns of failures caused by forces such as lateral wind pressures, earthquakes, or punching shear of slabs. Since the foundations transfer the structural loads such as vertical dead and live load and lateral wind load to the ground at the bottom of the structural frame, foundation failures will typically *telegraph* their effect *upward* for the full height of the structure. Such failure also may affect the stability of the high-rise structure and often may result in a noticeable tilt of the structure. This is especially so in cases of unequal support. (See Figures 8.1 and 8.2.)

8.1.2 Foundation Failures

The foundation is the structural element providing support for the various loads acting on the structure. It is the link between the structure and its eventual support, which is the soil itself. The actual transfer of load may be by direct bearing on soil or rock or by intermediary elements such as piles or caissons.

When we speak about foundation failures, we often refer to both the failure of the structural elements of the foundation such as footings or piles and the failure of the soil itself. Whereas the first type of failure may be the result of overloads on the foundation or its understrength, the second type results from overconfidence in the borings or other subsurface information or loss of bearing value because of adjacent work. Foundation failures resulting from failure of the footing itself is a rare occurrence. One such case was the fracture of the post-tensioned concrete foundation mat in Washington, D.C., which was caused by a design error. (See Figures 8.49 to 8.51).

The foundation is usually a human-made structural element, whereas the soil is a material found in its natural state, disturbed or undisturbed by humans. Artificially made soils are also used in construction (these are mechanically compacted soils or soils made totally from nonnatural materials). The construction of a foundation introduces new conditions into the soil. This happens as the subgrade is exposed and, therefore, unloaded, as changes in the internal friction of the soil are effected by blasting or by the densification of soil by pile intrusion.

Ground conditions often vary considerably from one location to another within the confines of a single construction site. Sometimes the variation is so great that, when two borings do show' identical soil layers, the validity of all borings will be questioned. Rock exposures may range from fine gray syenite, hard seamy limestones, granites, to schists of various hardnesses. These vary to such an extent that they are ringing hard or so soft that a pipe pile will penetrate over 10 ft (3.0 m) before indicating a 30-ton (267-kN) resistance. In some areas even seams of serpentine and asbestos are found.

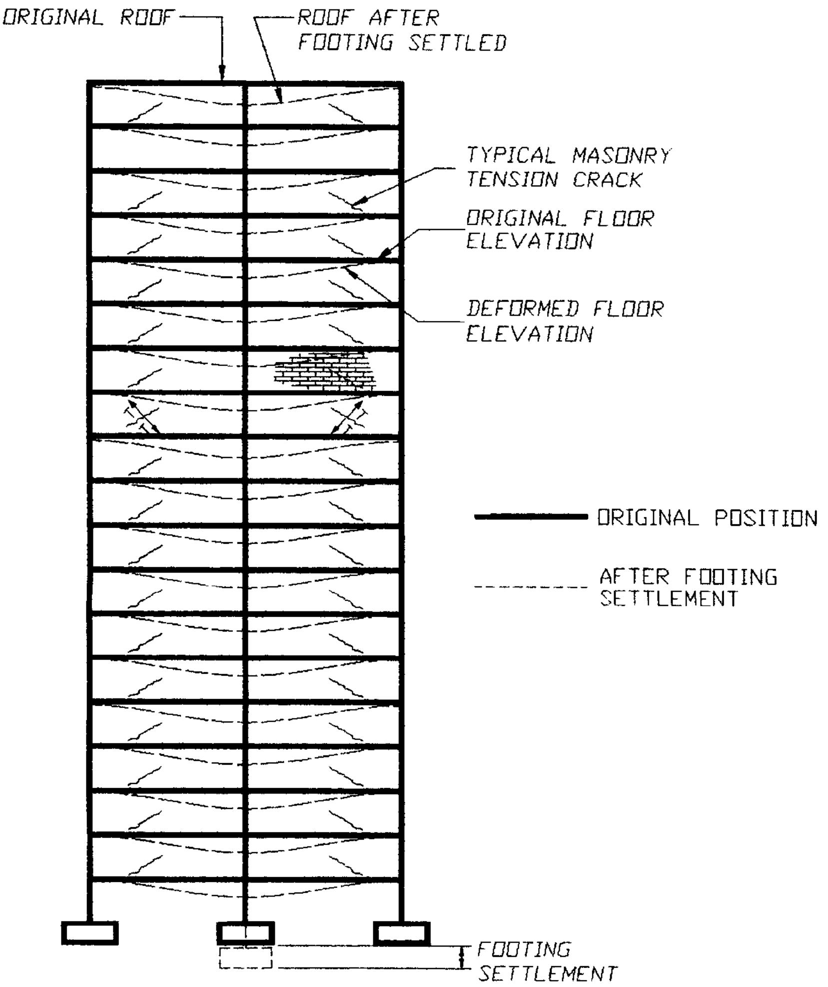


FIGURE 8.1 Settlement of center column—high rise—showing effects of settlement of center column telegraphing upwards of the roof of the high-rise structure.

However, granular materials will range from hardpans to uniformly sized, running fine sands. Soils also often contain clay deposits and organic silts. And, of course, there are the exposed and buried mud deposits with organic layers intermixed in a series of lenses to considerable depths. Because natural soils are often so varied and inconsistent, difficulties in foundation design and construction are expected and are encountered, often with associated trouble.

Actual *water* level in rock excavations sometimes has no relation to the level indicated by borings or to conditions next door. A typical example was the completed work for the New York Lincoln Center underground parking facilities and mechanical plant, where groundwater was first encountered at quite a high level. This water later disappeared after some anchorage drill holes were completed; it was later discovered almost 20 ft (6.1 m) lower than the groundwater encountered in the Philharmonic Hall, adjacent to it.

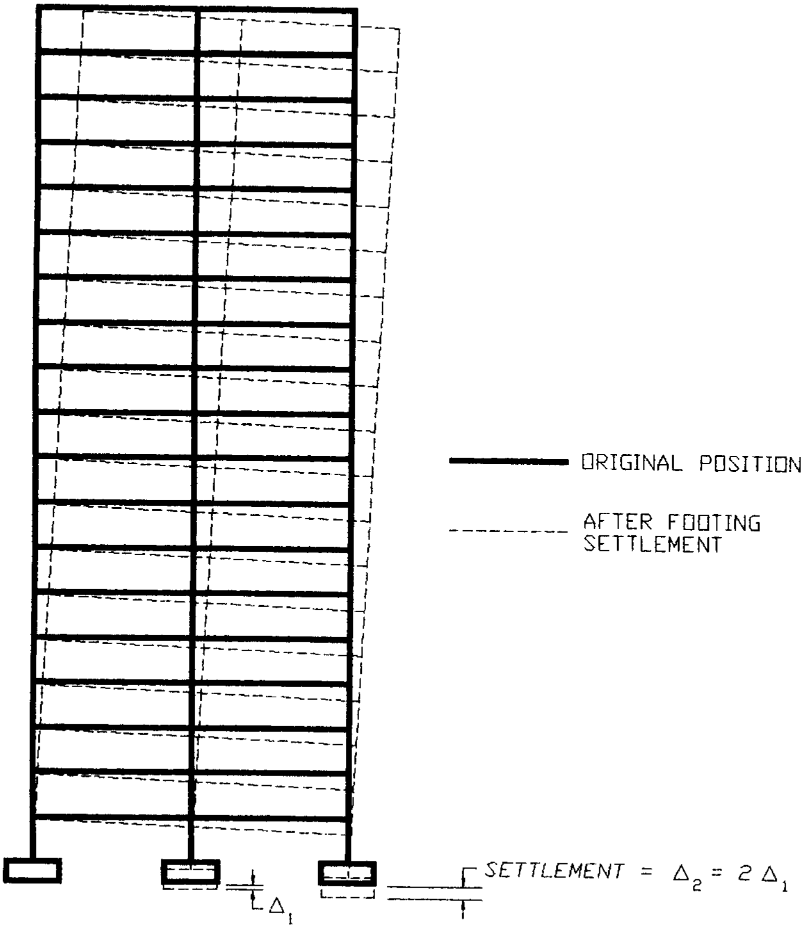


FIGURE 8.2 Structural tilt—a result of footings settlement—showing high-rise structure tilted sideways as a result of proportional footings settlement.

Bed rock surfaces are often very erratic and irregular. During the excavation for the Sixth Avenue subway in New York City during the 1930s, it was found that a strip only 100 ft (30 m) wide was 40 ft (12 m) lower on one side than on the other.

During the construction of the new Bellevue Hospital in Manhattan in 1960, the main hospital building was supported on piles as long as 100 ft (30 m). The adjacent parking facility, which was located no farther than 50 ft (15 m) away, was founded on concrete piers bearing directly on rock.

It is not uncommon to find large vertical mud holes or wide gaps filled with silt within a rock structure.

With such erratic rock strata, rock blasting has a serious effect on the behavior of rock. Rock seams must be found, traced along their length with the seams cleaned and packed and tied across with grouted dowels and rock anchors. Rock as a supporting subgrade can be made safe and sound, but only with careful planning and special work.

Granular soils are found in different varieties. There are some very good and some not-so-good granular soils. Some dense consolidated glacial sandy gravel deposits can safely sustain 10 tons per square foot (957 kPa), whereas uniformly fine grained and very loose silty sands are treacherous and extremely difficult to control. Glacial deposits also vary. Sometimes old sand fills are taken to be natural deposits. Standard borings contribute little information to distinguish the man-made from the natural geological sand layers. Only actual load tests are effective indicators, which often will show surprising looseness of these fills.

Layers of varied *silts and clays* cover many rock troughs and old shore lines. These layers are quite stable when left alone but become liquid when disturbed. Dewatering must be done slowly, to permit the seams to drain out, otherwise "boils" and collapse of braced sheeting excavation will result. Pile driving also generates local liquid conditions that flow readily. The plastic flow of the shore lines continuously pull the river structures outward. Even structures supported on piles carried to rock are affected and rotate with time.

Buried layers of *peat* are sometimes interspersed with silt or sand deposits and are found at great depths. Where piles receive their support at soil layers overlying peat deposits, serious settlements are common. Also, where pumping occurs in areas where footings are bearing above peat layers, settlements are common.

Pumping of water out of weak soils is known to cause consolidation of these soils and result in settlement damage to existing structures. Several precautionary methods are presently being employed to avoid this damage. Water-tight sheeting enclosures with deep cutoffs and internal pumping has been often found to be an adequate solution.

It is well recognized that all loads must be transferred to the underlying soils so that the resulting settlements can be tolerated by the structure without distress. At the same time, stability must be maintained over the life of the structure.

The ten most common categories of foundation failures are:

1. Undermining of safe support
2. Load transfer failure
3. Lateral movement
4. Unequal support
5. Drag down and heave
6. Design error
7. Construction error
8. Floatation and water-level change
9. Vibration effects
10. Earthquake effects

A foundation failure obviously is a serious event, since such a failure may trigger the collapse of the entire structure. This is critically so because most structures are based on structural systems whereby the support of the upper levels always depends on the structural integrity of the lower elements. Ironically, some foundation failures have been economical successes. One such example is the Leaning Tower of Pisa in Italy (Figure 8.3). A close look at this tower will reveal that the tower started to lean during its construction. The reason for this conclusion is a slight change in the slope of the tower, near its midpoint. It is an indication that the builders attempted to correct the foundation-settling problem (Figure 8.4).

7.2. UNDERMINING OF SAFE SUPPORT

Robert Frost in his poem "Mending Wall" (published in *North of Boston*) describes the troubles with stone walls in New England and concludes, "Before I built a wall I'd ask to know what I was walling

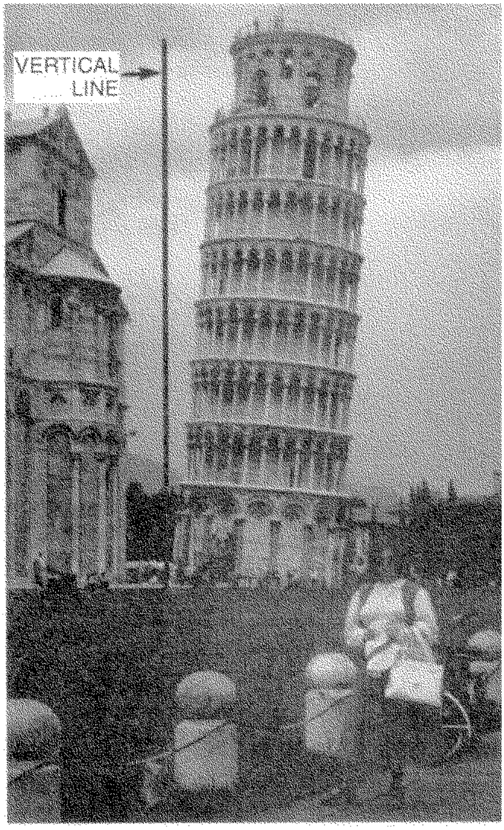


FIGURE 8.3 Leaning Tower of Pisa.

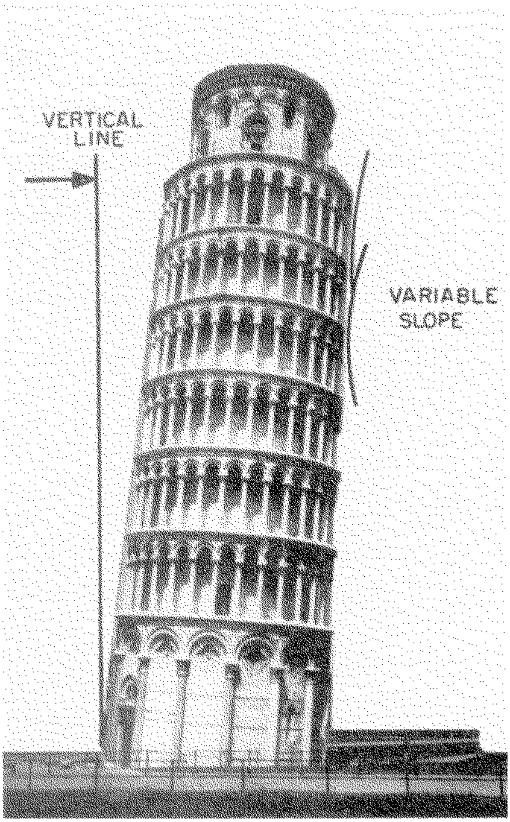


FIGURE 8.4 Leaning Tower of Pisa—variable tilt.

in or walling out."[1] To paraphrase this warning, before a foundation is designed, one should know *what loads are to be carried and what soils are expected to carry the loads.*

The need for a thorough soil investigation prior to the undertaking of a construction project cannot be overemphasized. In addition to the careful study of the soil strata directly below the proposed structure, existing adjacent structures must be reviewed with care. The need for temporary supports must be evaluated. A well-designed bracing and shoring system is often needed to prevent a lateral shift. A permanent support structure such as underpinning should be installed where the new construction will undermine an existing present support system. Where these provisions are ignored or entirely omitted, serious distress follows (Figure 8.5), and sometimes tragic consequences result (Figure 8.6). Another example is the total collapse of a five-story building on 34th Street in New York City that occurred when the vertical support was lost as a result of loss of lateral restraint. This happened when the excavation for a new high-rise building came too close to the foundation of the existing old building. Fortunately, the collapsed building had been evacuated a short time earlier because of unsanitary conditions. As a result of the collapse, the rubble filled the cellar completely and piled up almost to the second floor level.

Excavations for new sewer trenches adjacent to existing buildings have frequently caused undermining of footings resulting in distress and even total collapse. In the President Street collapse in Brooklyn, and similar other collapses, several occupants of existing buildings were killed or injured

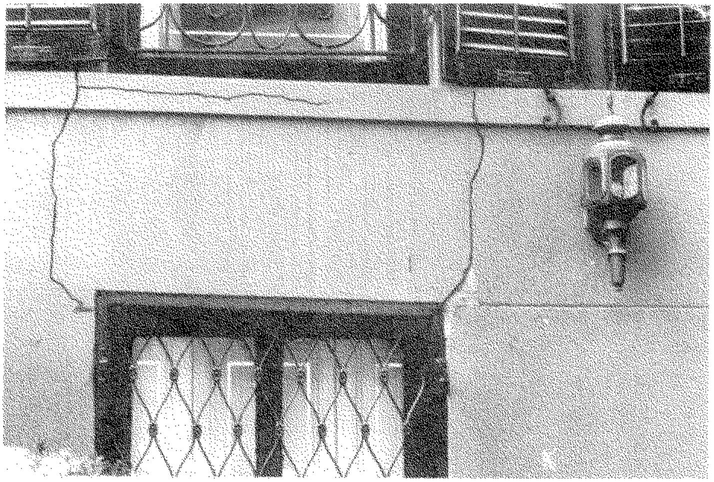


FIGURE 7.5 Structural damage caused by adjacent excavation.

FIGURE 7.6 Building collapse caused by adjacent excavation.

when their buildings were undermined. When these excavations were close to the existing footings, and within the influence line (Figure 8.7), the footings were undermined, often with disastrous results.

8.2.1 New York Suburb Sewers

Construction for a new sewer main project in Brooklyn required the excavation of deep cuts adjacent to existing old residential buildings. The sewer lines were designed to run too close to existing footings. Not only were the new cuts within the influence lines of existing footings, but the contractor used vibratory pile drivers to install the piles. There was no monitoring of the vibration levels during pile driving. As a result, many nearby buildings settled, cracked, tilted, and shifted laterally. After the damage occurred, diagonal braces were added for lateral support (Figure 8.8).

One building collapsed, killing one of the tenants (Figure 8.9). The investigation revealed that the building was constructed using timber footings, a construction method common at the turn of century. The footings were found to be in good condition but had been disturbed by the sewer activity. Once the excavation was within the influence line, vertical support was lost, causing footing set-

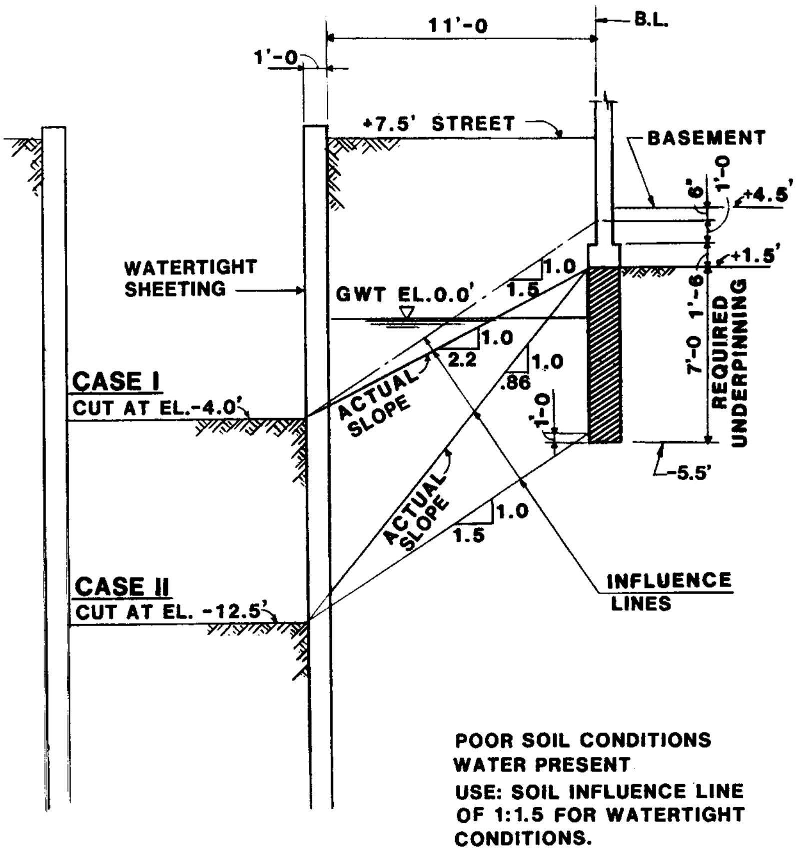


FIGURE 8.7 Bracing structures for lateral support.

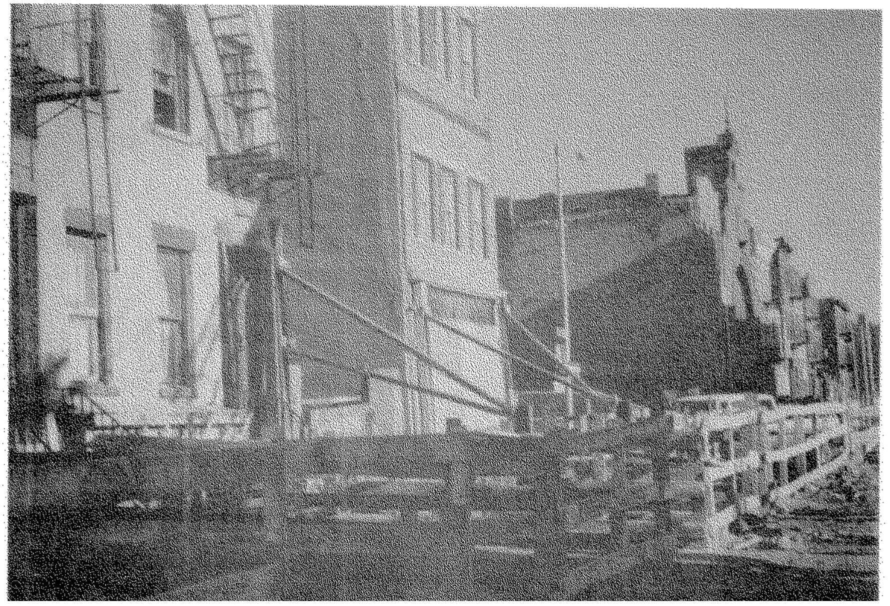


FIGURE 8.8 Bracing structures for lateral support.

FIGURE 8.9 Building collapse—ineffective bracing.

tlements. The high-grade steel bracing system (Figure 8.10) could not save this building, because soil support was lost *below* the footing level.

To restore the integrity of the vertical support it was initially decided to underpin the buildings. The bid prices obtained for underpinning were so high, however, that it was more cost effective to buy the damaged buildings and demolish them.

Lessons to be learned:

1. Careful *preconstruction* study to determine the need for protection and underpinning of adjacent structures is required. This study should review existing plans, taking sufficient soil borings and evaluating the condition and strength of existing buildings and other structures.
2. Excavations for sewers or other utilities should be moved away from existing buildings and so located that the excavation cut is outside footing influence lines.
3. Use of vibratory equipment for pile driving should be avoided in loose sands and similar soils.
4. Vibration readings, using seismographs, should be taken as often as needed to establish the suitability and proper energy of the driving hammers.

8.2.2 Electronics Plant, Upstate New York

An electronics plant was under construction when the entire building started to sink vertically and slide laterally toward the low adjacent valley. Instrument survey readings confirmed these visual observations. The footing movements, as is usual in these cases, were accompanied by distress in the building in the form of cracking of slabs and concrete grade beams (Figure 8.11).

The main question that had to be answered was what was the effect of large movements of the order of 4 in (102 mm) on the structural steel frame and its connections (steel bolts)? To answer this question, we had to determine the level of stress in the loaded and deformed steel beams and the ac-

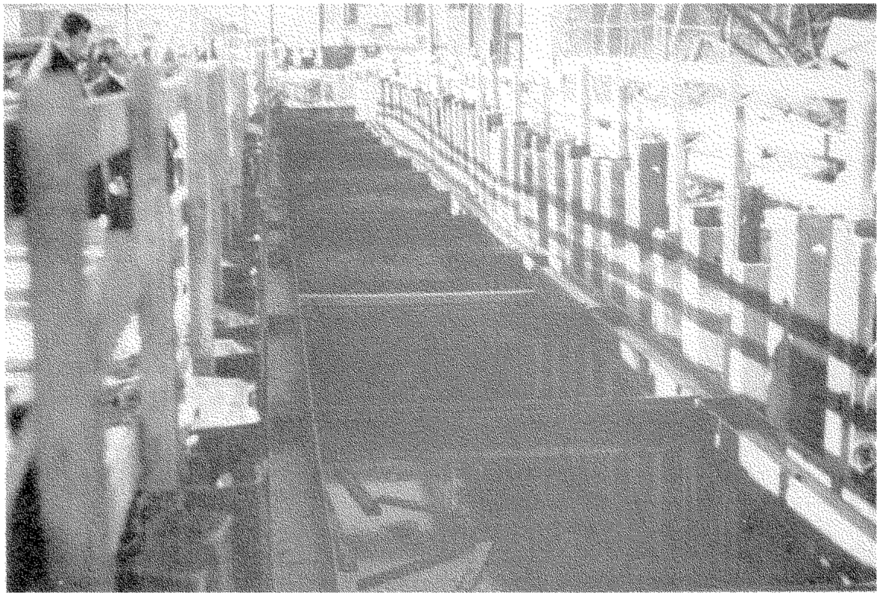


FIGURE 8.10 Bracing excavation cut.

FOUNDATION FAILURES AND REPAIR: HIGH-RISE AND HEAVY CONSTRUCTION

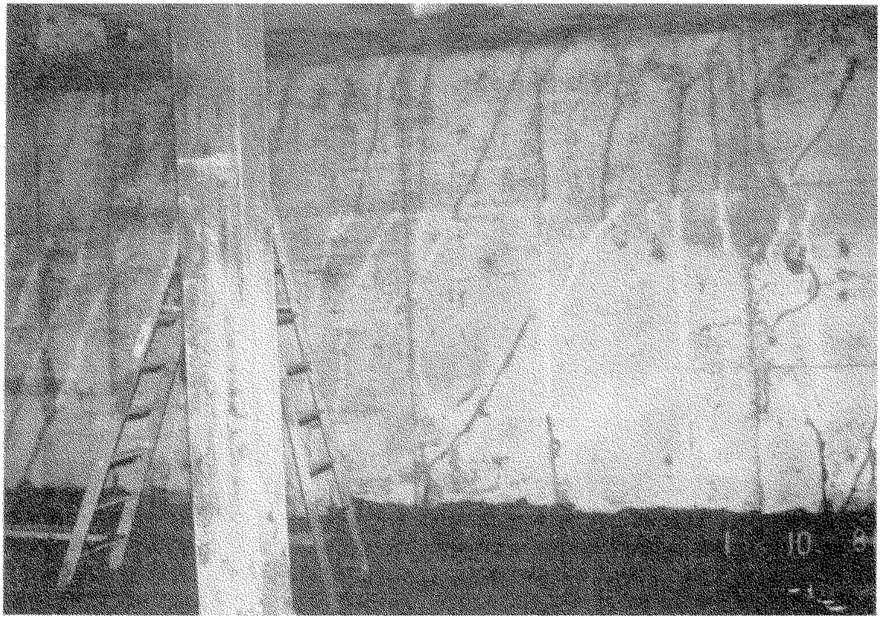


FIGURE 8.11 Structural failures (cracking) in deep-grade beams.

tual loads in the connection bolts. For that purpose we used the innovative stress-relief method (Figure 8.12). This method, which our firm pioneered, proved that the level of stress actually present in the steel and its connections was low and acceptable.

This procedure requires the attachment of electrical strain gages to the steel flanges of the beams. Through each strain gage, a hole is then drilled directly into the steel. The reduction of stress at the hole is measured by the strain gage connected to a Wheatstone Bridge. The relieved stress thus measured gives the approximate stress level then existing in the loaded steel beam. The only structural corrective measure that was executed was to enlarge the existing footings, so as to enable the structure to support the additional live loads without considerable settlements.

The method of repair is shown in Figures 8.13 and 8.14.

Lessons to be learned:

1. High footings adjacent to severe soil slopes should be designed using low allowable bearing pressures to reduce possible settlements.
2. In extreme cases, the use of retaining walls should be considered.
3. The actual stress level in a deformed structure is the true yardstick for evaluating the adequacy of a distressed structure.
4. The stress-relief method proved to be an effective tool in strength evaluation of distressed structures.

8.2.3 East Side Hospital, New York City

During excavation for a new high-rise hospital structure, an alarm went out to everybody connected with the project. It was discovered that, as a result of rock removal by the foundation contractor, the adjacent high-rise apartment building to the west was precariously sitting on a "sliver" of rock (Fig

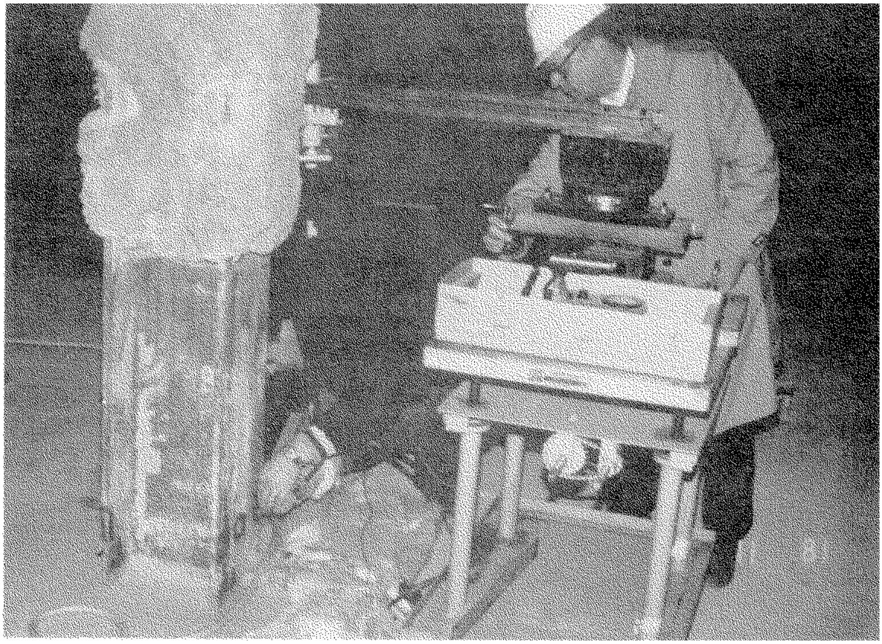


FIGURE 8.12 Stress-relief test.

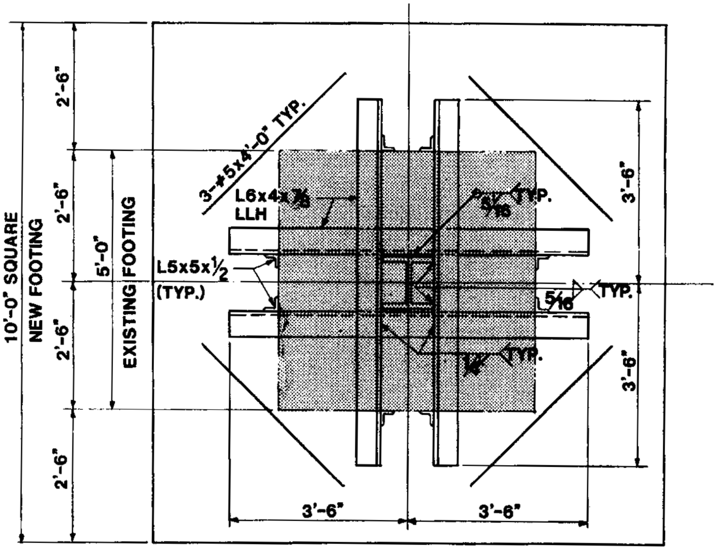


FIGURE 8.13 Increased footing size to accommodate live loads.

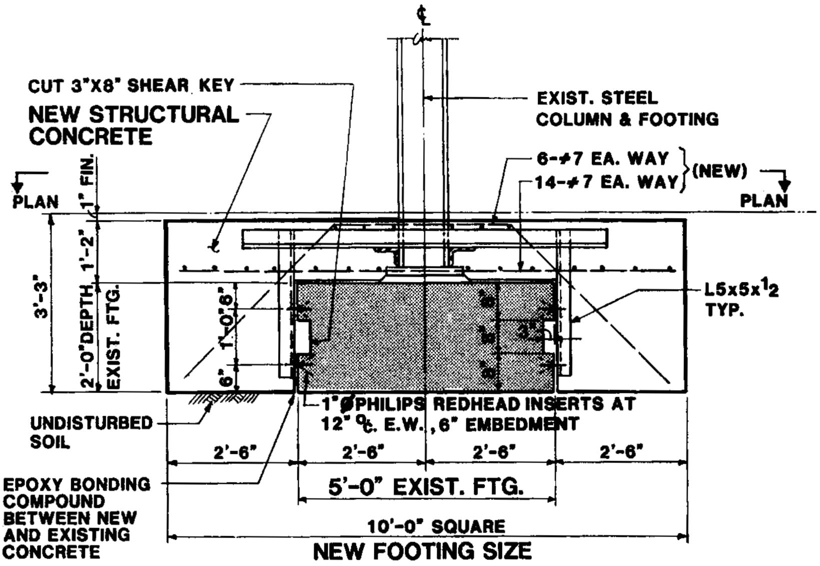


FIGURE 8.14 Cross-sectional view of Figure 8.13.

8.15). Construction work came to a halt, and serious consideration was given to evacuating the fully occupied apartment building.

Probes into the wall revealed the following conditions (Figure 8.16):

1. The width of the rock "sliver," of unknown strength, was approximately 14 in (355 mm). Incredibly, it was the only structural element supporting the high-rise building.
2. Even more remarkably, the footings called for in the original design for the existing apartment building were not built at all.
3. The easterly wall of the building cellar was not a standard concrete cellar wall but was actually a rock face with stucco finish.
4. The plans that had been filed with the Department of Buildings did not show the "as-built" conditions.

An emergency plan for underpinning was immediately launched. This plan included new concrete underpinning piers (braced laterally) cut into the rock "sliver" and bearing on the rock below the existing cellar level (Figure 8.17). The plan was executed perfectly, and the alarm was turned off.

As usual, the responsibility for the unexpected additional cost was in dispute. The questions asked by everybody were:

1. Was the original 1964 construction, which deviated from the design, bearing the apartment building columns on the neighbor's rock, legal?
2. Did it follow accepted good construction practice?
3. What responsibility, if any, does the Structural Engineer of Record have for the contractor's variation from the plans?

Regarding the first question, our research into the building code showed that the code was completely silent on this issue. Regardless, we found no requirement forbidding the practice. Our opinion was that it is poor construction practice for one building owner to utilize a neighbor's rock for

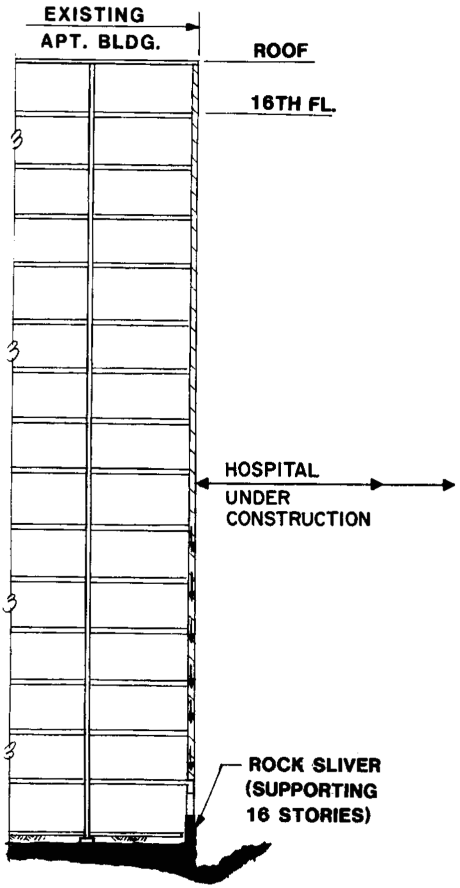


FIGURE 8.15 Rock sliver precariously supports 16-story brick wall.

bearing for the owner's building, especially since he or she has no control over nor any right to limit the neighbor's construction activity.

The Engineer of Record had no "Controlled Inspection" responsibility for this project, neither was he required to perform such an inspection of the construction according to the then-pertinent code.

According to many building codes, when neighboring construction goes below the bottom of an existing footing, it is the responsibility of the neighbor doing the new construction to underpin the existing structure, when his or her footings are more than 10 ft (3.0 m) below curb level.

The New York City Building Code, for example, states[2]:

> (b) Support of adjoining structures.—
>
> (1) Excavation Depth More Than 10 Ft. (3.0 m)—When an excavation is carried to depth more than 10 ft. (3.0 m) below the legally established curb level, the person who causes such excavation to be made shall preserve and protect from injury any adjoining structures. . . .

Lessons to be learned:

1. Do not use the rock outside your property for support of your structure, regardless of the apparent immediate savings.

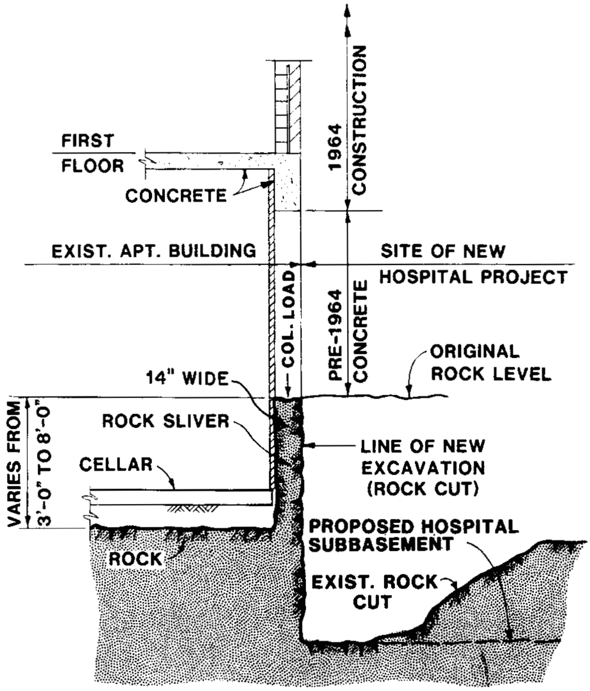


FIGURE 8.16 Cross section of Figure 7.15.

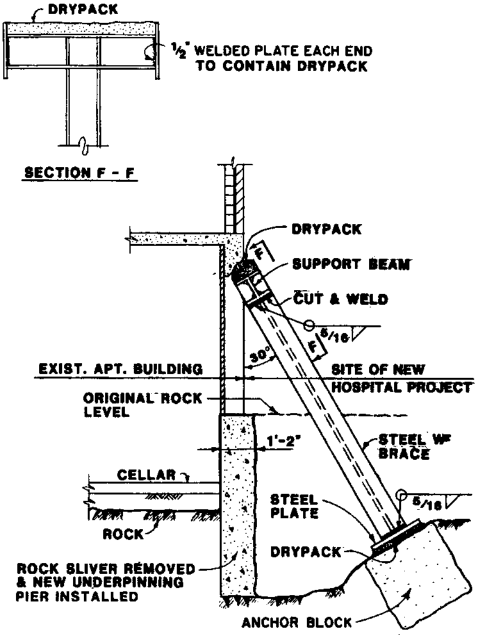


FIGURE 8.17 Emergency plan for underpinning existing building to create proper bearing.

2. You may support your structure only on the rock totally within your own side of the property line.
3. Rock "slivers" should not be relied upon for foundation support.

8.2.4 Manhattan Hospital Complex, New York City

When the structures adjacent to a deep excavation begin cracking, it slowly starts to sink in that something is wrong. An excavation extending an entire site was progressing toward a total depth of 45 ft (14 m) below grade. The existing structures contained only single-level basements. Initially, it was expected that much of the excavation would be into sound rock, which was identified as gneiss. The ground-water level within the buildings was controlled by a sump pump located below an existing basement slab.

During the design phase it was obvious that the adjacent structures would have to be underpinned. The New York City Building Code requires that underpinning will be designed by a professional engineer. This requirement was met, and a design using underpinning pits with post-tensioning cables to resist lateral earth pressures was devised.

As excavation for the pits was proceeding, movement was detected in the building directly east of the site. This movement was recorded by the tell-tales (movement gages) which were properly installed for monitoring such movements. As water was found several feet below the pits, attempts were made to dewater the soil surrounding the pit excavation. But, as recorded by the inspection engineer, ". . . this proved futile because of the impermeable nature of the soil existing at this location."

Dewatering by well points was then tried. This also proved ineffective, because the silts in the soil clogged the well point screens. A third alternative using jack-piles was then considered. This method of pushing pipe segments into the ground requires the use of the building weight to act as a reaction for the jacking forces. In this project, unfortunately, it readily became apparent that the footings of the existing building, which consisted of decomposed rubble, did not offer sufficient resistance to the jacking loads.

Even an attempt to use a steel beam to jack against failed, and the scheme was then abandoned. While these efforts were going on, a further adverse movement was detected in the adjacent building. At this time the safety of the structure was questioned, and temporary rakers (sloped braces) were installed. For all practical purposes, construction operations came to a stop.

The two buildings on the east exhibited serious cracking throughout (Figures 8.18 and 8.19). The one nearest the excavation settled, causing the entire building to pivot about the foundation walls. This pivoting effect in turn caused the entire party wall to move westward, leaving a 6 in (152 mm) gap at the top (Figure 8.20). This movement caused enormous distress on the interior of the building. Floors settled, ceilings and walls cracked, and even the elevator got stuck as its shaft deformed and would not permit free movement of the cab. At the fourth floor there was actually a separation between the wood joists and the party wall, causing the floor to settle by as much as $1^1/_8$ in (28 mm) at one point.

The foundation contractor, realizing the danger, immediately installed vertical shores under the west end of the wood joists, from the first floor all the way up to the roof.

It was at this point that our firm was retained. Subsequently it was recommended that the contractor take weekly movement readings using surveying instruments and continue doing so until the settlement stopped. The contractor was also instructed not to proceed with any restoration work until all settlement had stopped. *The New York Times* of July 3,1977, printed the following article regarding this project under the headline "When the Earth Opens and Walls Move[3]":

> George N. and Barbara K. don't live here any more. Their former apartment, in a brownstone in the Gramercy Park area, once looked like the home of many a professional couple—antique furniture, luxuriant plants and exposed wooden beams, lots of beams.
>
> . . . Their apartment is one of the occasional casualties of excavation work, the bulldoz-

FIGURE 8.18 Damage resulting from excavation.

FIGURE 8.19 Cracking of partition wall and settlement of stairs due to footing movement.

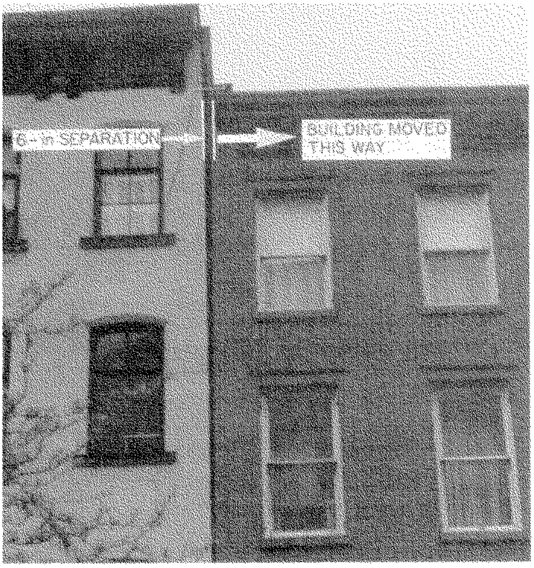


FIGURE 8.20 Lateral displacement of existing building toward excavation.

ing, blasting and drilling that create the cavernous holes meant to hold a new building's foundation.

Though New Yorkers frequently sue or arrange out-of-court settlements with excavators (claims range from apartment damage to sexual inadequacy caused by noise), George and Barbara just decided not to renew their lease.

. . . The whole experience was very unnerving, "Mr. N. recalls. "I was really concerned how much physical damage would be done to my home by the end of a day. It started out as a hair-line crack and got bigger every day (author's italics). I saw the baseboard move further away from its original place on the floor. Then we had to take down a shelf in the closet because it was no longer wide enough. *The wall had moved four inches (102 mm) toward New Jersey.*" (author's italics).

. . . A side of the ceiling was now without support and, just as Mr. N. had feared, it soon gave way. Fortunately, their apartment was saved by beams, which had been installed by workmen a few days earlier. [The vertical shores referred to above (author's comment).]

. . . Dr. F., a physician, and her husband, resident-owners of the building next to the excavation, never contemplated selling their property.

Although the roar of the machinery can reduce telephone conversations to the F. residence to frustrating shouts, Dr. F. says that somehow her practice at home (psychiatry) "has not been disturbed."

. . . Mr. R., project engineer for the excavation company, says that the unknown factor in that site was an unexpectedly high underground water elevation. . . . "We had what you call test borings. . . . As it turned out, these test borings were greatly different from what we encountered."

When all efforts to support the building failed, the method of last resort was used, very costly soil solidification by freezing the soil. The saturated soil proved to be an advantage for the freezing process. This process with all its pipes and equipment took about 3 weeks to install (Figure 8.21). It

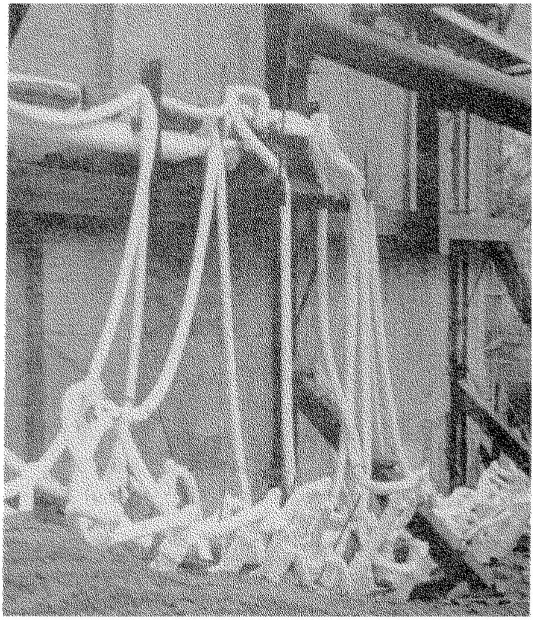


FIGURE 8.21 Soil solidification by freezing.

proved to be a total success, as practically all movements within the adjacent "ailing" structure stopped. Once the soil under the building was consolidated, the original conventional underpinning design was implemented. Just prior to freezing the soil, a cross-lot bracing system was installed (Figure 8.22). The purpose of this system was to eliminate any further movement in the building on the east and to stabilize it during the subsequent construction operations. The giant cross-lot braces that spanned the entire lot permitted construction to proceed without interruption. After completion of the underpinning, the adjacent buildings were restored.

FIGURE 8.22 Cross-view of Figure 8.21.

Lessons to be learned:

1. Accurate preconstruction subsoil study is essential to the success of foundation work. Inadequate test data can cause serious delays, cost overruns, damage, and even collapses. In this case, had accurate information been obtained, design changes and soil solidification could have been planned prior to start of construction.
2. Adjacent structures must be carefully monitored during underpinning operations, so if and when movements are detected, changes in methods or procedures can be implemented.

8.3 LOAD-TRANSFER FAILURE

A well-designed and constructed rigid-frame structure will tolerate even substantial foundation movements. When an assembly of walls, floors, frame, and partitions is rigidly connected, the system will adequately adjust itself to differential foundation movements. The load transfer is made by frame action through the support offered by the foundation. When this interconnected rigidity is absent, the load transfer will be through a single support so that the load will go directly to the soil vertically. Where this single support in the soil is missing, the structure will fail, unless the structural rigidity will transfer it horizontally to other footings and then, in turn, to the soil.

Where the adjacent rigidity is present but lacks sufficient strength, the adjacent structure will fail. Figure 8.23 shows a diagrammatic description of this action. Four men are carrying a log, its weight uniformly distributed. When A steps into a ditch (an inadequate foundation), his portion of the load is suddenly transferred to B, who may be unable to support the additional load.

One such typical example was a 16-story warehouse building in lower Manhattan, supported by four continuous pile caps and timber piles. Pumping operations at a neighboring 20 ft (6 m) deep excavation for a depressed roadway lowered the water table and caused one line of piles to settle and pull the pile cap down.

This released the exterior support so that the load transfer to the interior line of columns overloaded their steel beam grillages. The 24 in (610 mm) beams collapsed down to a 12 in (305 mm) depth with consequent 12 in (305 mm) settlement of every floor in the building. The floor beams

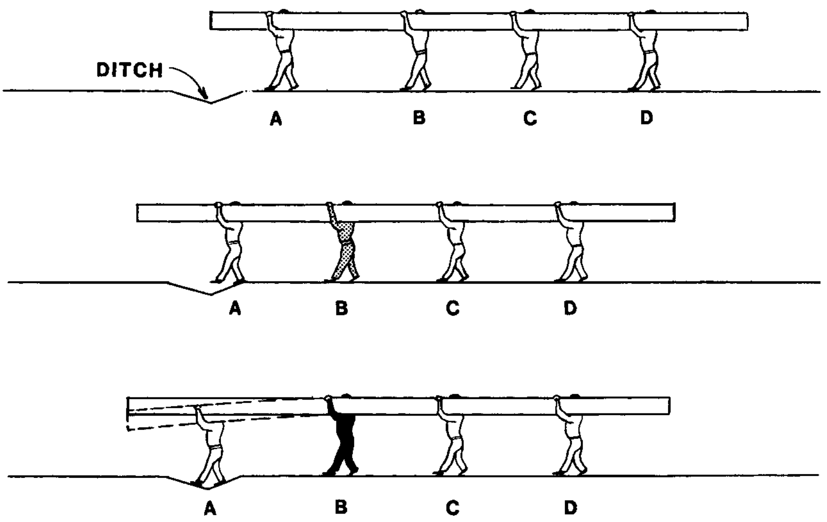


FIGURE 8.23 Diagram of log deflection.

resting on steel girders between columns had only "soft" support so that they took the new trough shape without failure.

The entire building was shored, and new shallow grillages were installed to replace the failed ones. Rather than jacking the entire structure, which entailed definite risks, especially for an old structure, each floor was releveled, and the building was put back into use.

8.4 LATERAL MOVEMENT

It is well known that 1 inch (25 mm) of lateral movement of a foundation causes more damage than 1 in (25 mm) of vertical settlement. Lateral movements are caused from either the elimination of existing lateral resistances or from the addition of active lateral pressures and loads. The changes in the active pressures and passive resistances as a result of variable water conditions must be considered. Saturation of the soil often increases the active pressures and reduces the passive resistances. Many collapses have occurred as a result of demolition of adjacent buildings. In these instances the backfilling of adjoining cellar space generated lateral soil pressures on the old cellar partitions. These walls, in turn, failed because they were not designed and constructed as retaining walls.

Lateral flow of soil under buildings is also known to cause collapses of buildings. In Sao Paulo, Brazil, such lateral soil flow caused the total collapse of a 24-story office building in 1943.

A similar case occurred in 1957 in Rio de Janeiro, also in Brazil, where an 11-story residential building collapsed completely after the excavation of an adjoining wall removed the lateral restraint of the building piers. The attempted underpinning, which was started too late, could not save the rotating structure.

There also are numerous cases of wall failures from broken drains alongside the footings with washout of the soil during heavy storms.

Change in pressure intensity against walls often causes failure, especially in the unreinforced concrete basement walls for residential buildings. These walls are unfortunately seldom investigated for high soil pressures. Surcharging soil on land adjacent to structures often causes large lateral pressures. Mounds of debris from demolitions are frequently piled adjacent to basement walls that had not been designed to resist such loads. These bowed basement walls often cave in and cause the total collapse of the structure.

The stability of retaining walls is often similarly affected, resulting in failure by overturning, either because of inadequate base width or because of high soil pressure caused by defective drainage or adjacent load surcharge.

Failure of buried sanitary structures are also common, especially when they are emptied for cleaning or repairs.

8.4.1 New Jersey Postal Facility—Cracked Pile Caps

Shortly after completion of the construction of this mail bag repair shop, the entire structure started to shift sideways, causing many of the piles to bend and the pile caps to crack (Figure 8.24). In addition, an interior block wall parallel to the direction of the shift also cracked (Figure 8.25).

Our firm was called to investigate and to determine the structural adequacy of the entire facility. What was immediately evident was the very heavy vertical gravity loads imposed on the piles. Figure 8.26 shows heavy bales of paper and cloth sitting directly on the already shifted and distressed piles.

The area directly east of the plant was found to be surcharged with mounds of heavy garbage dumping (Figure 8.27). It quickly became clear that the soft clay soil directly underneath this extreme garbage weight was surcharging the soil, causing lateral pressures on the long, unbraced steel piles. Previous attempts to "hold the line" by welding steel cross-ties had been unsuccessful, and rather than stopping the lateral shift, they only transferred the lateral loads to the adjacent piles, which eventually failed, too. The additional lateral cross ties may be seen in Figure 8.28.

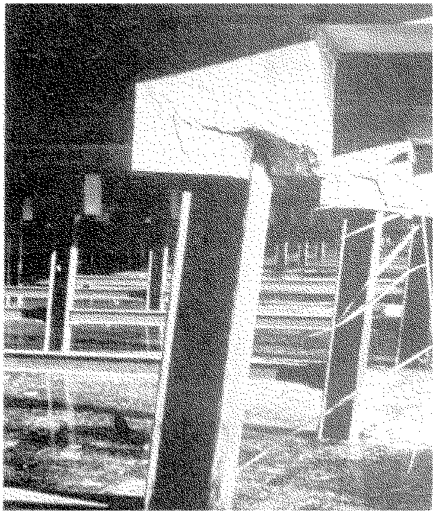


FIGURE 8.24 Lateral failure of H piles and concrete caps.

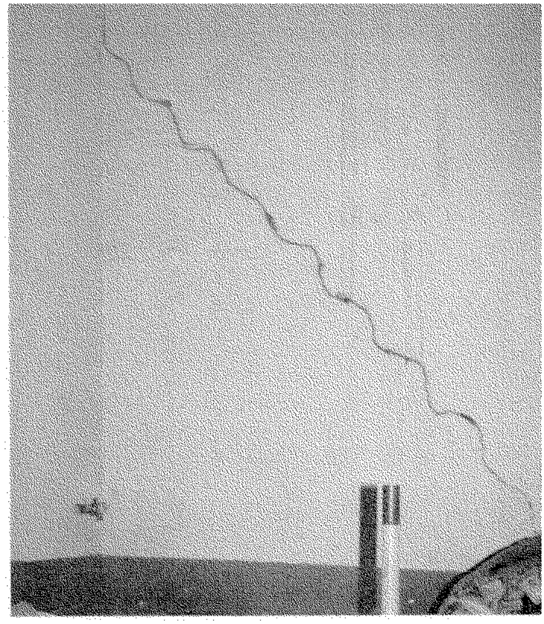


FIGURE 8.25 Diagonal masonry cracks due to lateral and downward settlement.

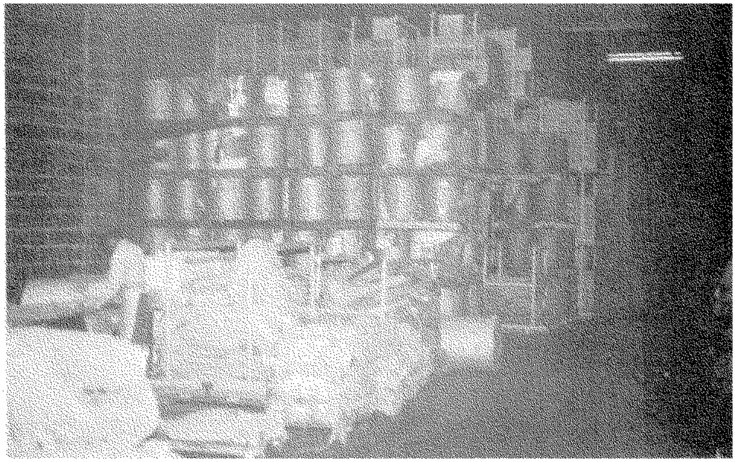


FIGURE 8.26 Unsafe loading on damaged piles (Figure 8.24).

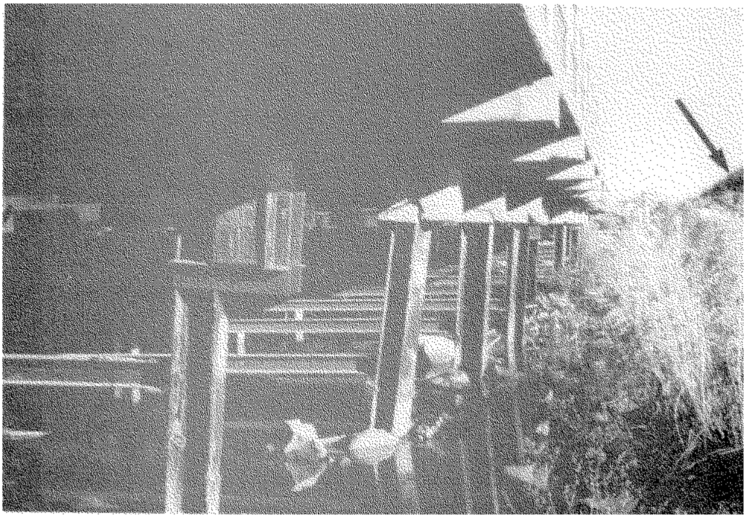


FIGURE 8.27 Lateral soil shift causes pile failure.

We immediately recommended that the owners reduce the actual live loads on the piles, by moving the heavy bales to unaffected areas. A complete underpinning program involving drilling through the existing slabs and jackpiling down through the soil, was then recommended. The owners dragged their feet for a while, but finally a program similar to one proposed by our firm was implemented several years later.

Lesson to be learned:

The placement of high surcharge adjacent to structures generates high lateral loads that can cause shifting of piles and bowing of foundation walls, and should be avoided.

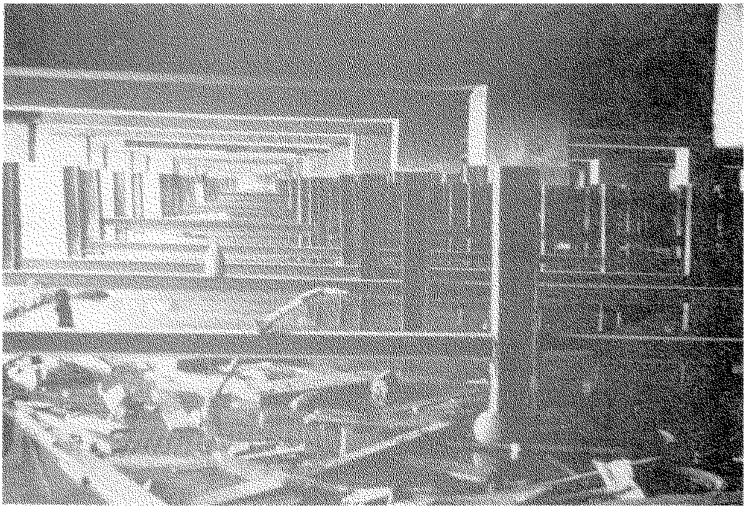


FIGURE 8.28 Steel cross-ties stabilize piles.

8.4.2 The Collapse of the Hartford, Connecticut, Foundation Walls

This large housing development is located on a sloping terrain, using concrete foundation walls enclosing the basements. The walls were unreinforced, common for low-rise residential buildings. Rather than using controlled granular fill, the contractor used clay backfill during a dry period of weather. This clay backfill shrank away from the concrete walls, leaving narrow gaps along the full height of the walls. After a heavy rainstorm, the runoff filled these gaps, causing very high hydrostatic lateral pressures. Some of the walls which were not yet braced at the top toppled over, and some bent, cracked, and completely failed [Figures 8.29(a), 8.29(b), and 8.30].
Lessons to be learned:

1. Foundation walls must be braced at top and bottom prior to any backfilling.
2. Only well-controlled and drainable, porous backfill should be placed against foundations and other retaining walls to avoid shrinking soils and subsequent development of high lateral soil and water pressures.

8.4.3 Long Island Water Pollution Control Plant Outfall

During 1972 and 1973, precast concrete sections were installed for the Long Island Water Pollution Control Plant Outfall. Later, in April of 1974, during a dye test conducted to check the continuity of the ocean outfall, it was discovered that there were open joints along the bay length. Further investigation disclosed that 2300 ft (710 m) of pipe sections out of the overall length of 22,000 ft (6706 m) were either displaced, damaged, or completely separated.

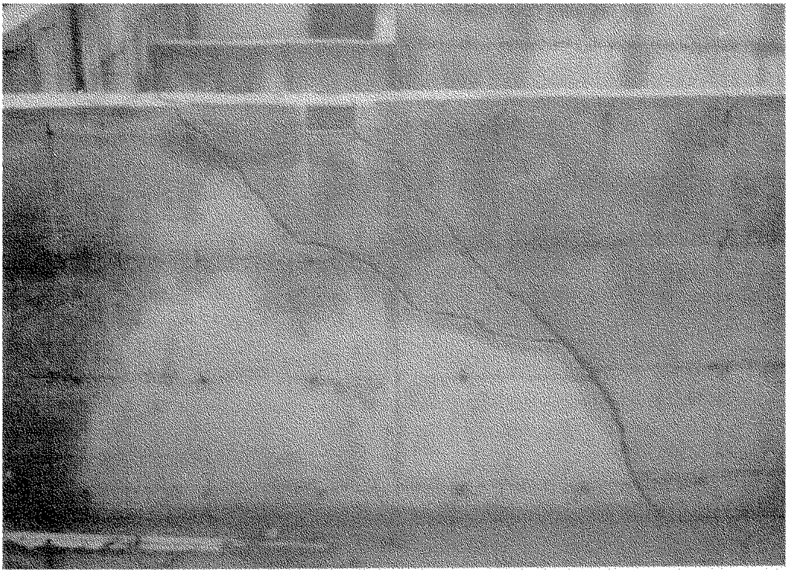
The outfall consisted of an 84 in diameter, prestressed concrete pipeline pipe sections, 20 ft (6.1 m) long, which were generally assembled into 100 ft (30.5 m) strings and installed on pile supports spaced at intervals of 20 ft (6.1 m). The outfall consisted of three sections: a 17,000 ft (5182 m) bay section which was essentially horizontal; a 5000 ft (1524 m) section sloping at a 3% grade; and the remaining horizontal section. The failure occurred in the sloping portion, which deformed into a lateral S-shaped form.

This S shape was consistent with a buckling failure of a compression member under high stress. These types of failures may occur under several basic conditions:

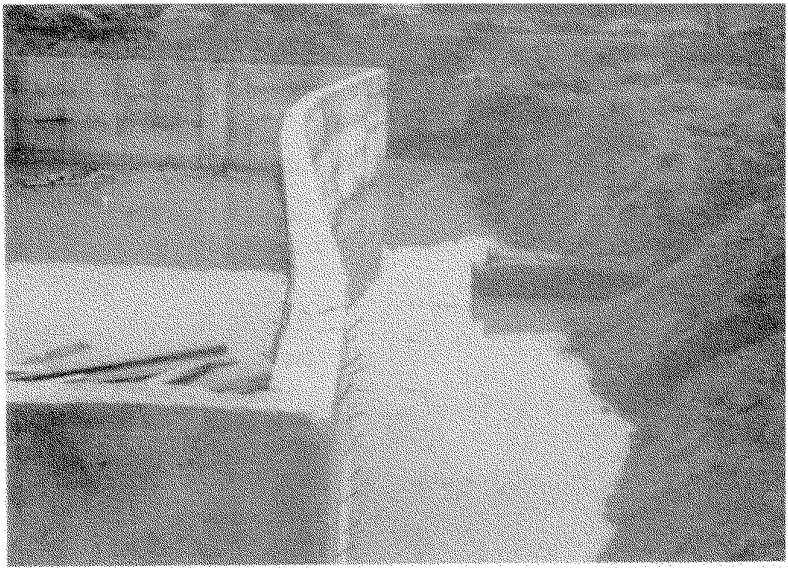
1. Structural continuity
2. Axial compressive force
3. End restraints
4. A slight eccentricity with/or a lateral load adequate to trigger the initiation of the failure

Our review of all the data indicated that all the above-listed conditions were met in the summer of 1973.

1. The pipeline was completely closed, and all the sections were assembled and laid into the water under winter conditions.
2. The change in water temperature within and without the pipeline, which was estimated to be at least 30°F (–1°C) between February and August, induced the expansion of the concrete pipe.
3. End restraint was provided by the pile caps.
4. A small lateral soil pressure was considered to be the trigger most likely to have started the movement. The S bending shape was governed by lateral loading conditions. Movement downward was resisted by the pile support, and movement upward was counteracted by gravitational force. Therefore lateral movement was met with the least resistance. Additionally, uneven backfill will result in horizontal pressure component resulting in an unbalance sufficient to trigger the movement.



(a)



(b)

FIGURE 8.29 Total collapse of foundation walls [Figure 8.29(a) and (b)].

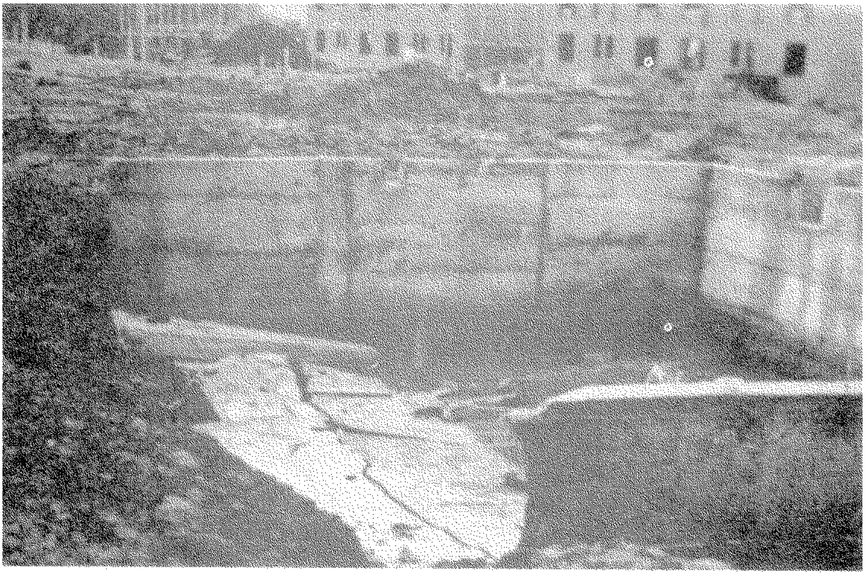


FIGURE 8.30 Total collapse of foundation walls [Figure 8.29(a) and (b)].

Lessons to be learned.

1. Special care should be taken to provide balanced lateral pressure by providing continuous quality control of backfill operations at underwater structures.
2. Although underwater structures are subjected to a rather small temperature differential, some relief must be provided to permit linear expansions.

8.5 UNEQUAL SUPPORT

A basic rule of engineering is that there is no load transfer without deformation. When loads are transferred to the soil through the foundation, this transfer is associated with a deformation of the soil. In other words, all footings settle when they are loaded. The amount of settlement is equal for different footings when soil resistances are identical and load distributions are equal. When they are not, differential settlements occur, and load transfer or tipping of the structure will result. The portion of the structure founded on the weaker soil will tip away. Where the framework is not continuous, the brittle masonry enclosure will crack (usually in a diagonal pattern) during the shear transfer. There are a great number of such structures, with foundations partly on rock and partly on soil, partly on rock and partly on friction piles, partly on stiff soil and partly on soft soil. There are no doubt many structures with footings bearing on different soils with different and unequal soil bearing resistances. The results are invariably distress and severe cracking (Figure 8.31).

All these soil support deficiencies may be corrected and are often so rectified. However, these corrections are usually quite costly and involve underpinning the weaker soil or the weaker support. The use of additional piles or the installation of jack piles are some of the most commonly used and most successful methods. The famous Transcona grain elevator in Manitoba, Canada (Figure 8.32) showed unequal settlement and tipped. This structure consisting of 65 concrete bins 93 ft (28 m) high on a concrete mat 77 × 175 ft (24 ×53 m), settled 12 in (305 mm) and then tipped 27° when

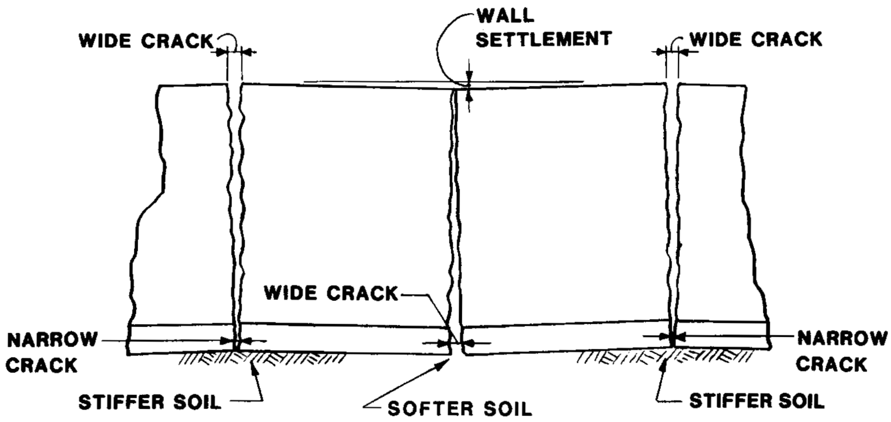


FIGURE 8.31 Cracks indicate rotation from vertical (heave) or settlement movement.

about 85% full of grain. The structure weighed 20,000 tons (178 MN) and contained at the time grain weighing 22,000 tons (195 MN). Already when repair work was begun in 1914 the massive structure, which was 34 ft (10 m) out of level, was jacked back to use with underpinning to rock—a remarkable engineering feat.

Sometimes, other methods of soil stabilization by cement or chemical injection are used. One other correction procedure utilizes the addition of a subsurface enclosure (usually a tight sheetpile cell) that increases the bearing value of the soil. This was done on a grain elevator in Portland, Ore-

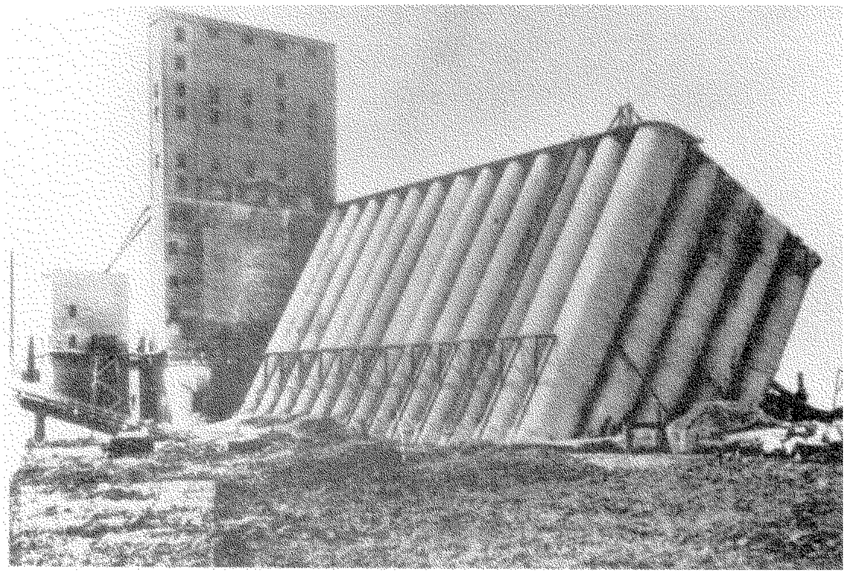


FIGURE 8.32 Tilt in Canadian grain elevator.

gon, in 1919, where 2600 pinch piles were added to enclose the foundations after the structure showed progressive unequal settlement and was tipping.

Not so fortunate was a smaller grain elevator in Fargo, North Dakota, which tilted in 1955 when it was only 1 year old. This elevator, which contained 20 bins 120 ft (37 m) high, was supported on a concrete mat measuring 52 × 216 feet (16 × 66 m). As the mat tilted, the bins crumbled and the structure was wrecked and had to be abandoned.

There are many classical examples of tilting buildings as a result of unequal settlements. The Tower of Pisa is probably the best known and researched example and is still in use, as is the Palace of Fine Arts in Mexico City (Figure 8.33) with its settlements measured in meters. The Guadalupe Shrine and Cathedral, also in Mexico City (Figure 8.34), is so distressed, cracked, and tilted that it makes visiting tourists nervous.

Buildings partially on earth and partially on rock have too often been designed on the incorrect assumption that the mere use of local building code allowable bearing values will ensure equal settlement. Where such variable bearing conditions exist, a separation joint must be provided so that each section of the building can then act as an individual separate structure. These buildings, especially when they utilize bearing walls, often crack almost immediately where such a joint is missing. Thus, nature provides the omitted joint (Figure 8.35). Underpinning and other repair costs together with the additional economic loss caused by delayed or vacated use far exceed the cost of properly designed and constructed separated foundations.

Our firm has also investigated several instances where soil shrinkage due to local dewatering triggered differential settlements even though the original construction was proper and followed faultless design.

8.6 DRAG-DOWN AND HEAVE

As a footing is loaded, the supporting soil reacts by yielding and compressing to provide resistance. The actual compressing of the soil takes place rapidly in the case of granular soils but much more

FIGURE 8.33 Settlement of the Palace of Fine Arts, Mexico City.

FIGURE 8.34 Foundation failure of Guadalupe shrine.

slowly for clays. Once this compression occurs, the structure remains stable, since the foundation no longer settles. This stability depends on fringe areas as well as the soil directly below the footing, or, in the case of piles, on the soil near the pile tip. If the soil below the footing is removed or disturbed, settlement or lateral movement is induced. Similarly, if the fringe area is removed, the soil reaction pattern must change. Even if sufficient resistance still remains without measurable additional settlement, the center of the resistance is changed, thus affecting the stability of the footing. When the

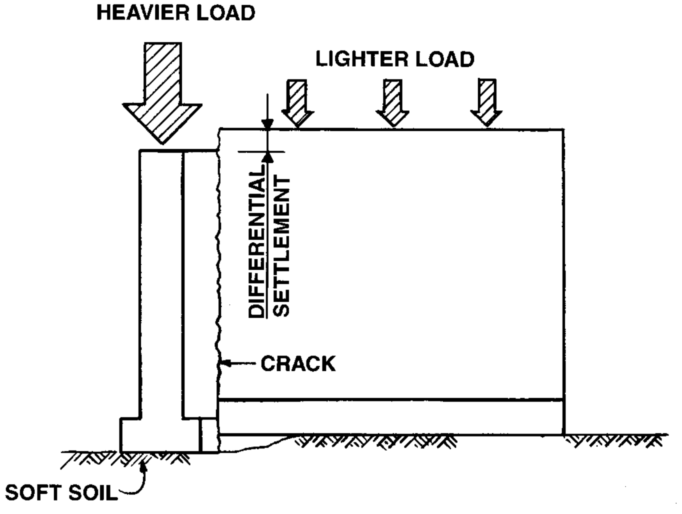


FIGURE 8.35 Differential settlement.

fringe area is loaded by a new structure, causing new compression in the affected soil volume, the old area can be required to carry some of the new load by internal shear strength. In such a case, there will be an additional unexpected new settlement of the previously stable building. In the event the new building is not separated from the existing construction, the settlement caused by the new load will cause a partial load transfer to the existing wall, with possible overloading of the previously stable footing.

In plastic soils, these new settlements often are accompanied by upward movements and heaves (Figure 8.36) some distance away. The liquid in the soils cannot change volume, and every new settlement must produce an equal volume heave.

When an entire structure settles at a slow rate for a long period, these settlements may be acceptable as long as they are uniform. Large differential settlements will result in damage.

To disregard such elementary considerations is often the cause of much litigation stemming from cases where new foundation construction in urban areas suddenly places neighboring existing construction in danger of serious damage or even complete failure. Under several building codes (The New York City Building Code is one such example), if new construction requires excavation more than 10 ft (3.0 m) in depth adjacent to an existing foundation, all precautions to avoid damage to the existing conditions are the obligation of the new project. If the new excavation depth is 10 ft (3.0 m) or less, the obligation for protection falls on the owner of the existing building should its foundations be higher than the new footings.

In medium- to well-compacted sands, pile driving demands heavy power, and the vibration impulses can shake adjacent structures to the point of serious damage.

Drag-down from soil shrinkage when water tables recede or from desiccation by tree growth, with consequent differential settlements, is a phenomenon observed in many localities. The failure of a theater wall in London in 1942 was correlated with the growth of a line of poplar trees. In Kansas City, Missouri, 65% of the homes in one residential area were found to be affected by soil desiccation from vegetation growth in foundation plantings.

Piles embedded in soil layers that will consolidate from dewatering or from load surcharge are subject to overloading when the greater soil density increases surface friction and the new soil loads “hang” on the piles. This phenomenon is sometimes referred to as negative friction. The added load may cause considerable increase in settlement and may even pull the pile out of the pile cap or pull the pile cap free of the column or wall.

Dewatering operations are commonly caused by new construction or pumping by water supply companies.

In some instances, as consolidation of organic clay strata takes place, piles continue to sink with additional settlement of the structure. This was the case at the Queens Apartments in New York.

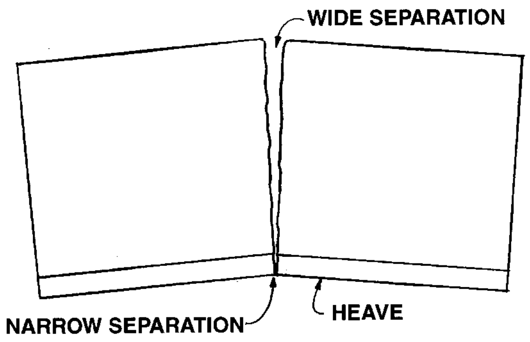


FIGURE 8.36 Upheaval.

8.6.1 Queens Apartments, New York

Settlements and masonry cracks developed in many levels of the buildings of this project (Figure 8.37). The cracks appeared in both interior partitions and the exterior brick walls.

These structures, located in Flushing, New York, were constructed over marsh land and were supported by timber piles. An earlier subsurface investigation of this project indicated that it was likely that at least some of the piles in the three-pile group supporting the distressed corner of the building had broken. This fact could not be confirmed without excavation or probes. The breakage may have occurred when the piles were driven or subsequently when the loads resulting from consolidation of the organic clay stratum were imposed on them.

In an effort to assess the potential for additional settlement and their likely magnitude, a laboratory and a field program were performed. In the laboratory, the rate of settlement due to a secondary compression of samples of the organic clay was measured. In the field, a sensitive settlement plate (Figure 8.38) was monitored for movement every 2 weeks for about 6 months, using a depth caliper accurate to 0.001 in (0.025 mm).

The area selected for installation of the plate was the interior corner directly outside of the laundry room. This area was chosen due to its close proximity to settlement cracks; it also provided sufficient overhead clearance for drilling.

A 1 in (25 mm) diameter heavy-duty pipe was driven into the lower sand stratum to provide a fixed reference point. A 2 in (51 mm) diameter outer casing protected this pipe from down-drag effects from the overlying fill and clay.

Analysis of laboratory data has suggested that settlement of the organic clay, as a result of the fill above it, can be expected to be about 1 in (25 mm) every 10 years. The field measurements from the settlement plate indicated even greater settlement.

Figure 8.39 presents the data obtained from the settlement plate device, with settlement rates of up to about ¼ in (6.4 mm) per year. Thus, the field data suggested settlements about twice those indicated by the laboratory data.

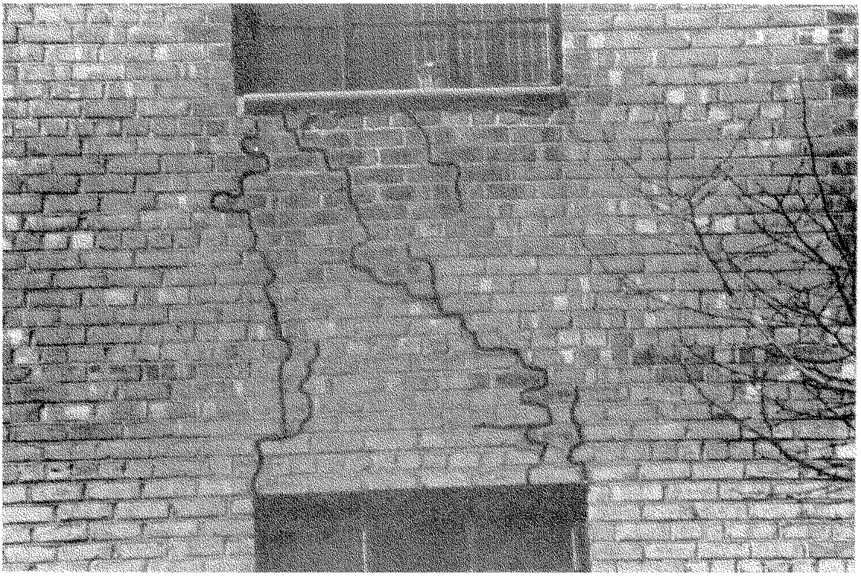


FIGURE 8.37 Masonry cracks due to settlement.

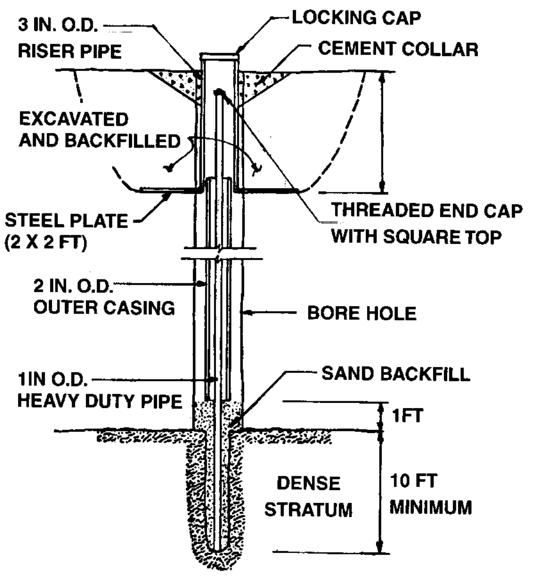


FIGURE 8.38 Instrumentation for measuring soil settlement.

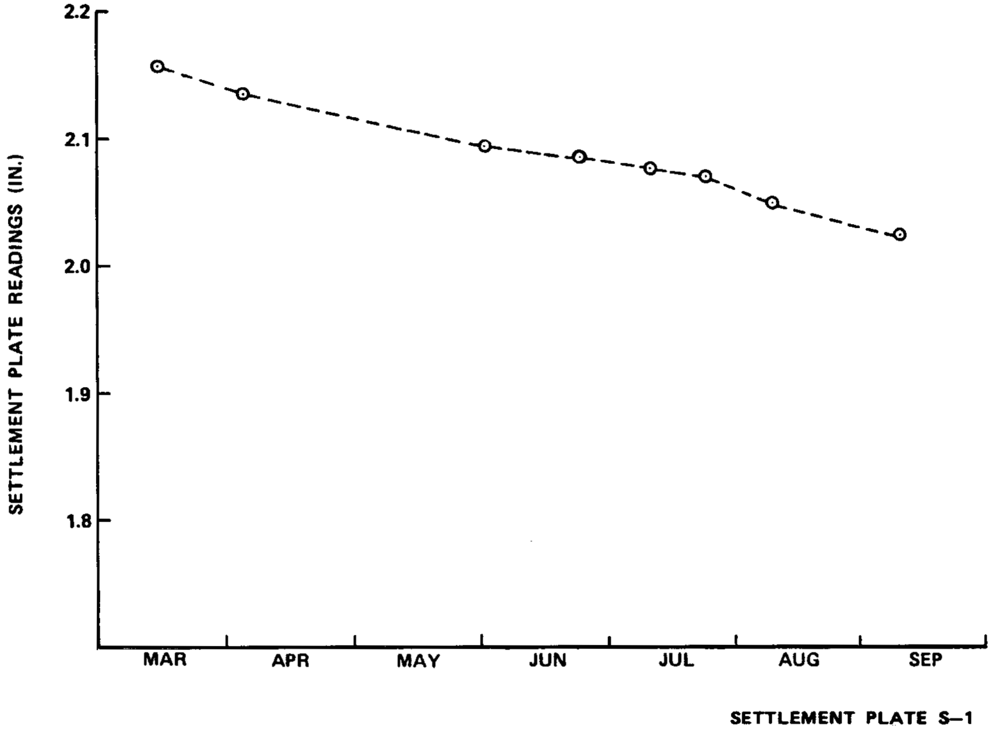


FIGURE 8.39 Chart of measured settlement.

Lessons to be learned:

1. Do not guess whether a structure that has settled will continue to do so. Instrument the foundations and the soil to assess potential future settlements.
2. Placing a structure on piles bearing in marsh land always involves risks of settlements.

8.6.2 Edgewater, New Jersey, High Rise

The piles supporting the first-floor slabs of this complex settled and pulled the slab downward. This caused settlements and cracks of the slabs and tilting and cracking of partitions (Figure 8.40). The first signs of distress were "sticking" doors and windows.

An in-depth structural investigation was carried out, revealing that although the main supports for the columns were bearing on steel piles driven to resistance in rock, the piles supporting the slabs were only wooden friction piles. The pattern of slab deflection was confirmed by instrument survey and the corresponding contour map (Figure 8.41).

Probes below the slab showed that the backfill had settled approximately 3 in (76 mm) (Figure 8.42). The negative friction effect of the downward settlement of the backfill and consolidation of the soil pulled the piles down. Consequently, the concrete slab attached at the top of the piles was dragged down.

The repair required cutting into existing slabs and adding new reinforced-concrete beams that transferred the slab loads into the reliable column piles (Figure 8.43).

Lesson to be learned:

When piles are driven into compressible soils, negative friction effects should be taken into consideration. Additional soil surcharge will increase the drag-down forces.

Heaving of footing contact surface is known to neutralize soil bearing by frost action. We have investigated several cases where during extremely cold winter, frost penetrated below slabs on grade

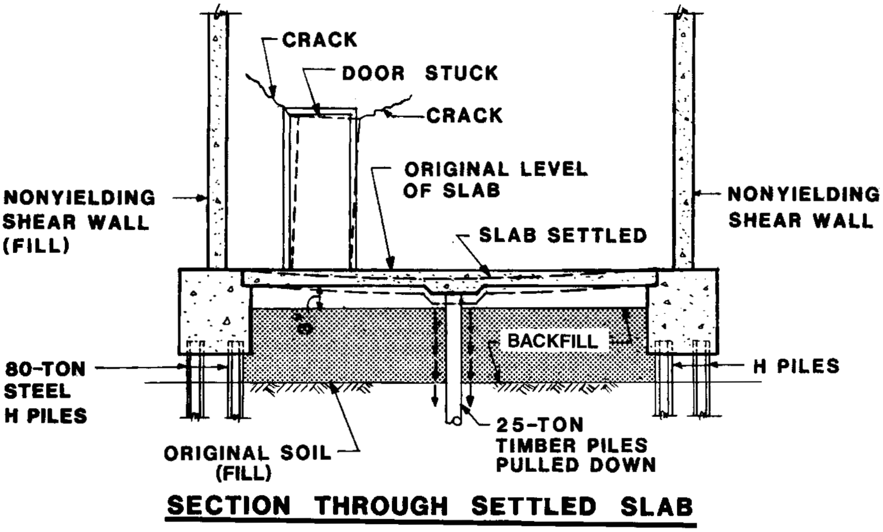


FIGURE 8.40 Structural distress relative to differential slab deflections.

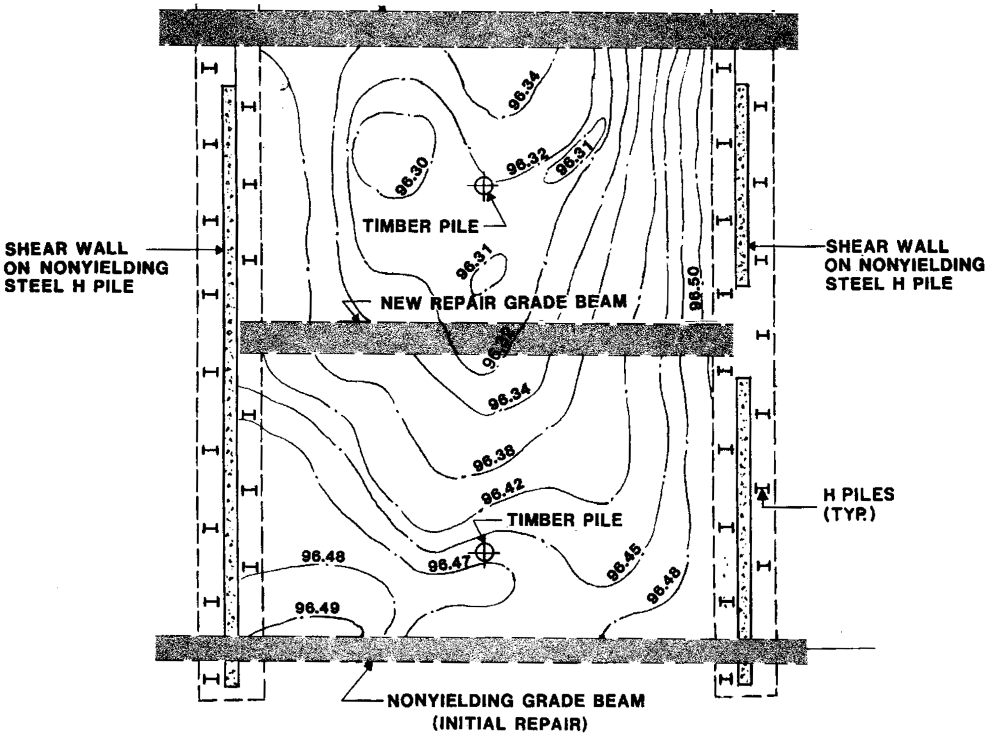


FIGURE 8.41 Slab elevations suggesting settlement of up to 2½ in (64 mm).

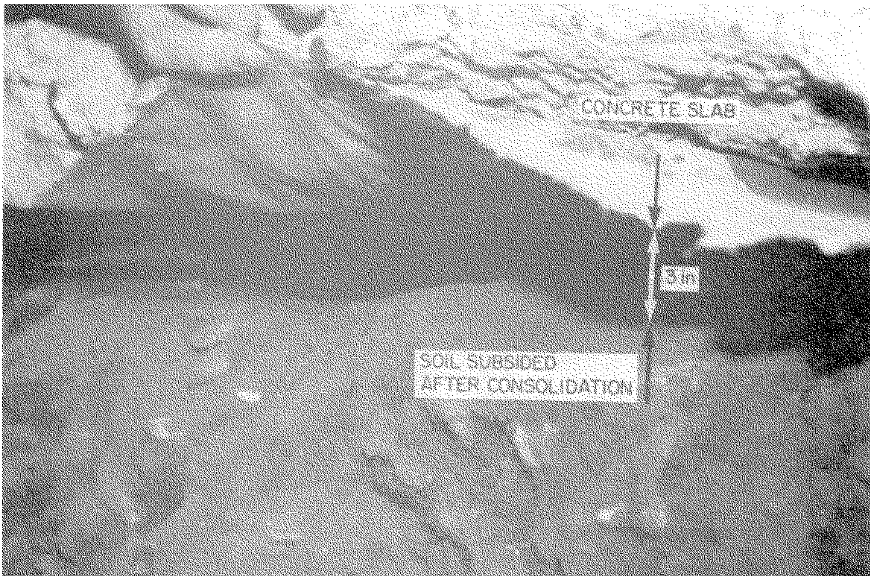


FIGURE 8.42 Settlement of fill leaving 3 in (76 mm) void.

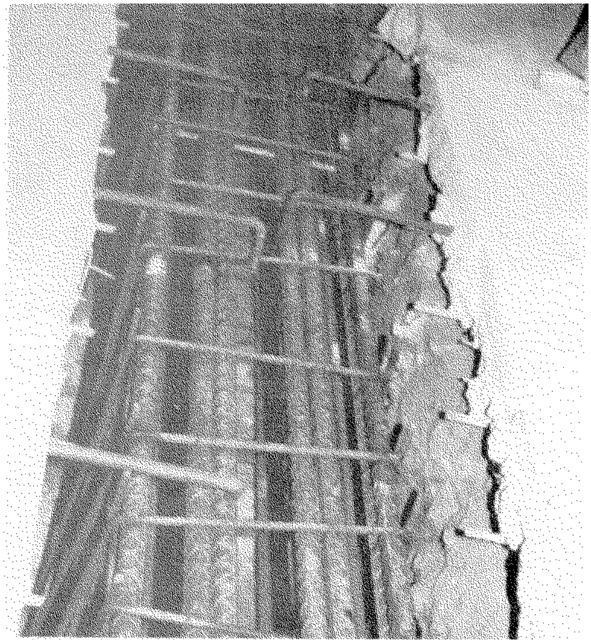


FIGURE 8.43 New concrete beams shift slab loads to steel plates.

and cracked buried water and sprinkler lines, causing ice buildup which heaved the structure above and even sheared some of the supporting columns.

8.7 DESIGN ERROR

Unfortunately, many foundations are designed with insufficient prior subsurface investigation or with disregard of the true soil conditions. This results in inadequate support for the structure, often requiring later expensive corrective work.

Standard structural design procedures for foundations will result in an adequate factor of safety; most errors are in judgment, leading to assumptions or decisions that are not consistent with the actual behavior of the soil later.

A common design error is often made, usually in an effort to save initial construction costs. In these cases the designers provide for pile support for the walls and the roof, while the main floor is placed on "more or less" compacted sand over fills, rubbish, peals, organic silts, and other compressible layers. Our own firm has seen many industrial plants where the roof was supported on piles while the floor supporting expensive equipment was placed over soil fill. It seems more logical to support expensive equipment on piles and let the roof settle. Heavy factory machinery generally requires installation on precisely level floors, and small differential settlements can become cause for concern.

Even in warehouse use, floor deformations become objectionable, making efficient use of forklifts impractical. Placing floor slabs on inadequate soil fills obviously is a false economy. True cost efficiency is measured by the total of the initial and in-life service costs.

8.7.1 The Alaska Pier Collapse

This unusual failure was caused by a design error resulting from inadequate preinvestigation study of local data and conditions. The formation of huge ice blocks atop the pier pilings was totally unexpected. The pier designers had not taken into account loads of such magnitude being imposed on the structure. The total collapse of the pier occurred at low tide when the total weight of the ice was suddenly suspended from the sloped piles (Figure 8.44).

There were unique conditions that contributed to this collapse. These were the extremely large variations between low and high tide, which were approximately 20 ft (6.1 m). The blocks of ice that had formed during the winter suddenly lost their support when the thaw came in the spring. The enormous weight of the ice was shifted to the piles, which bent, causing fracture of the concrete pile caps. The thawing of the ice at the contact surface with the piles also caused the ice blocks to slide down diagonally while being wedged between the piles (Figure 8.45). The bending stresses at the ends caused by the wedging forces (see force and moment diagrams shown in Figures 8.46 and 8.47) severely cracked the concrete pile caps (Figure 8.48) leading to the eventual collapse of the pier.

The author's files show that the partially completed pier was supported on pile bents spaced 20 ft (6.1 m) on center. The bents consisted of both vertical and diagonal piles. Twenty-inch octagonal prestressed vertical piles and 20 in (508 mm) diameter batter steel piles were arranged in pairs forming quasi A-frames. The length of the piles was between 70 and 80 ft (21 to 24 m). The piles were driven into 15 ft (5.0 m) of silt, then 55 ft (16.8 m) of silty gravel and sand underlain by layers of clay. By midwinter, huge blocks of ice, approximately 20 ft thick (6.1 m), had formed on the piles. The blocks weighing 50 tons (445 kN) or more slid down the sloped piles, damaging the piles and pile caps. The recorded telephone conversation between the design engineers on the project went like this:

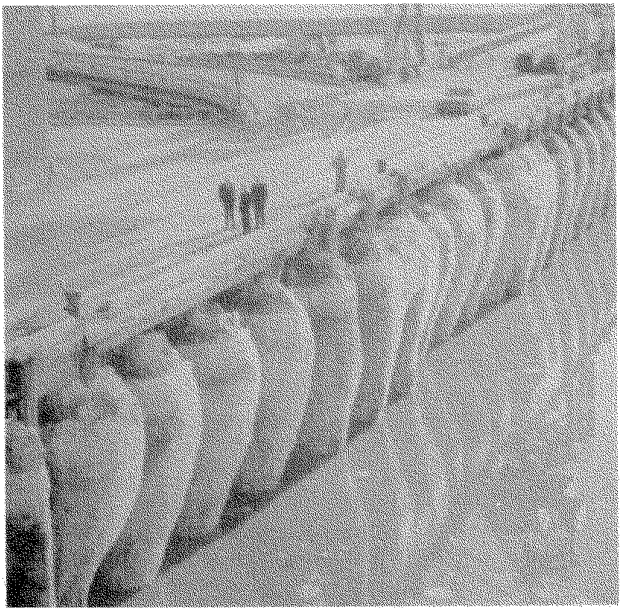


FIGURE 8.44 Ice-encrusted piles.

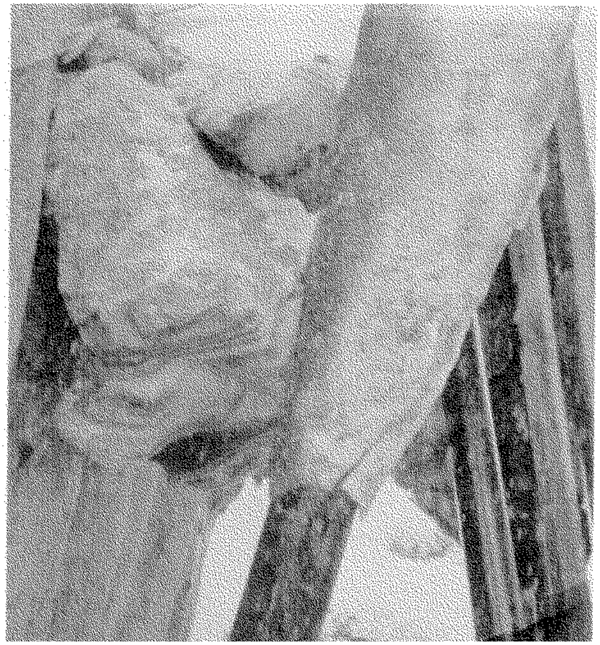


FIGURE 8.45 Ice melts, and blocks become wedged between piles (see Figure 7.45).

D: We've got great big problems up here . . . I went down the dock . . . a good half of our pile caps where the brace pile . . . are cracked.
They're cracked bad. They've got an inch and a half (38 mm) crack in some of these. We know it's because of the tidal action and the ice.

A: It looks like. the weight of the ice is causing such a moment up there that it is cracking the heck out of it. We're going to have to take the panels off, go in there and remove the caps and redesign the whole thing. . . .

D: We've got a good connection up there . . . it's very, very, rigid, but it's bending that's doing it. Would have saved us I think, if we'd had a lot more of those stirrups in there, but there are only No. 5 bars at about 18 in (457 mm) on center and it's just not enough.

While this failure was considered by many investigators to be caused principally by a design error, it was also established that damage was caused to the vertical piles during installation with overjetting and pounding with the diesel hammer. Insufficient penetration of the batter piles was also confirmed in several cases.

Lessons to be learned.

1. The effect of ice formation and resulting forces must be considered in the design of structures in northern exposures.
2. Piles subjected to lateral loading and bending must be sufficiently embedded into the concrete pile caps.
3. The consequences of extreme variations of tidal rise and fall should always be taken into consideration in design of marine structures.

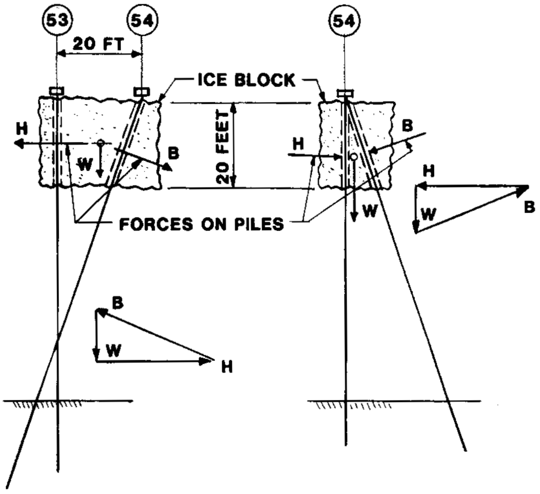


FIGURE 8.46 Lateral forces on battered and vertical piles due to ice action (see Figure 8.45).

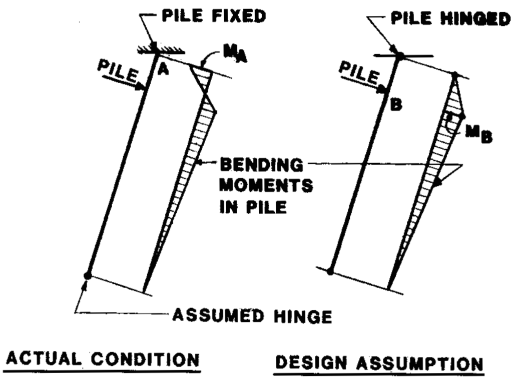


FIGURE 8.47 Lateral forces on battered piles due to ice melting (see Figure 8.46).

FIGURE 8.48 Failure of concrete pile caps due to ice action.

8.7.2 Washington, D.C., Foundation Mat

A 3 ft thick (0.92 m) foundation mat was cast with draped tendons (Figure 8.49) in accordance with the design requirements, and shortly afterward was post-tensioned. Unfortunately, as soon as the high prestress loads were applied, the slab lifted upward and cracked (Figure 8.50).

The error was discovered immediately because it was so obvious. It was amazing that the incorrect procedure eluded everybody involved with the project. The fact that the designer of the mat was a prestigious engineer only proves that no one is immune to mistakes.

As can be seen in Figure 8.51, there could be no equilibrium of forces in the foundation mat without the presence of the building columns. Realistically, the design should have planned for the prestressing to be executed in stages, with the prestress forces increasing as more and more weight of the upper levels of the structure was imposed. This procedure was not followed, and the failure ensued.

The repair was carried out using rock anchors to provide the necessary vertical reactions to the prestressing forces (Figure 8.52). The mat was repaired, and the prestressing was successful the second time around.

Lessons to be learned:

1. Posttension sequence must be carefully reviewed on a step-by-step basis.
2. Total equilibrium of forces must be ensured.

8.7.3 Boston Housing Slab Settlement

Where slabs on grade are expected to be at the same level as adjacent structurally supported slabs, total reliance on well-compacted backfill is merely wishful thinking (see Figure 8.53). Unless a de-

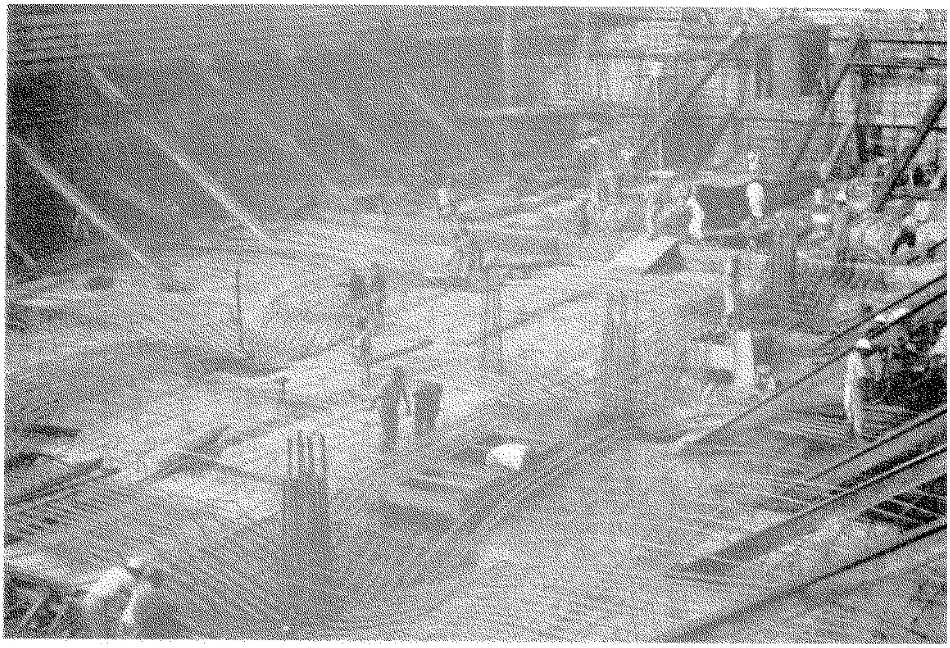


FIGURE 8.49 Concrete reinforcing prior to placement of concrete.

FIGURE 8.50 Prestress loads cause slab uplift.

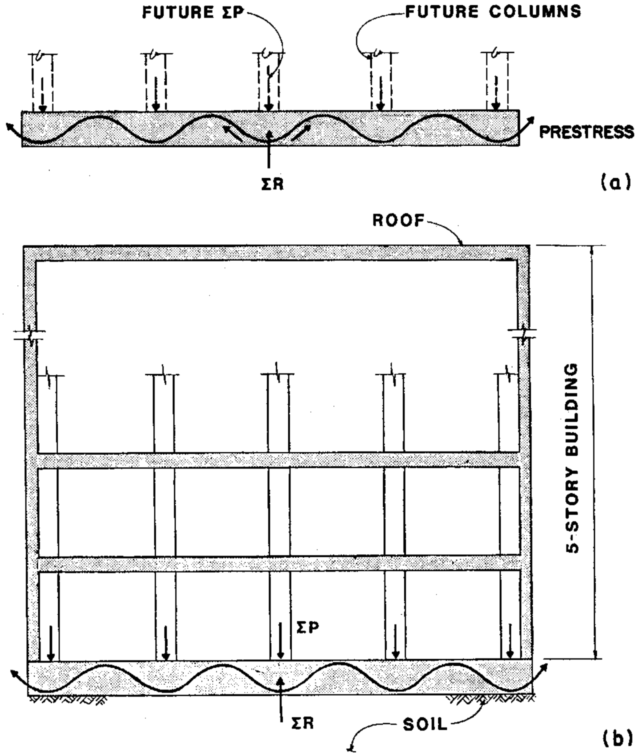


FIGURE 7.51 (a) Distress caused by design error in assumption of prestressing forces. (b) Lack of equilibrium of prestress forces is obvious.

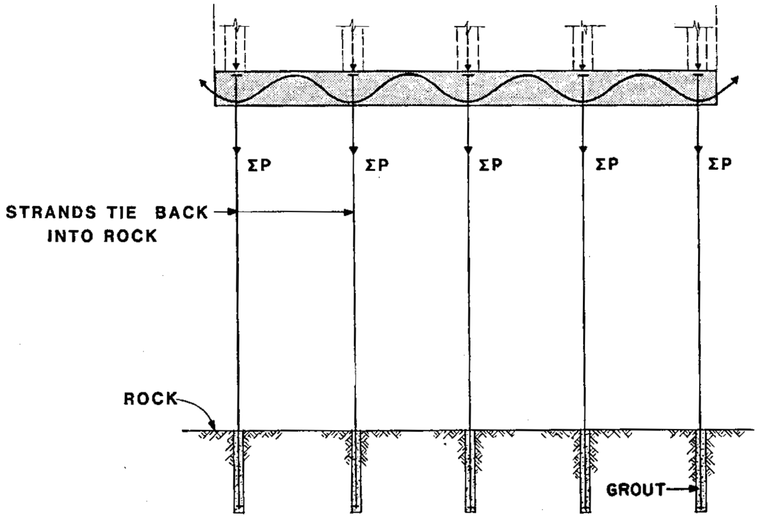


FIGURE 8.52 Rock anchors provide vertical reactions to prestressing forces (see Figure 8.49(a) and (b).

tailed compaction control program is set in advance of construction, an undesirable step will ensue as a result of differential settlement.

At the Boston project, the entrance to the 10-story building resulted in a hazardous step [painted red (Figure 8.54, see arrows)]. The proper design detail (Figure 8.53) will support the slab on grade (or fill) directly on a seat prepared in the foundation wall, thus preventing downward settlement.

Lesson to be learned:

To avoid steps caused by differential settlement, concrete slab on grade must be either cast on well-compacted fill or designed to bear on adjacent walls.

8.8 CONSTRUCTION ERRORS

There are two common types of construction errors:

1. Temporary protection measures during the construction phase
2. Foundation work itself

Most foundation failures involve the first type of error, relating to temporary shoring and bracings and temporary cofferdams for lateral protection and pumping operations. Because of the temporary nature of these structures, safety measures are often kept to a minimum for economy reasons.

Many construction errors on foundation work also are a result of the same deficiencies that exist in superstructure concrete work, such as improper concrete quality and improper rebar placement. Where deficient material is provided, in general there is absolutely no difference between superstructure and foundation work. The exceptions are foundation elements such as piles and caissons where “blind work” is involved. Cavities within cast-in-place piles are often found where quality control was lacking. The omission of verification methods early in the construction progress is partially responsible for the enormous losses caused by such errors.

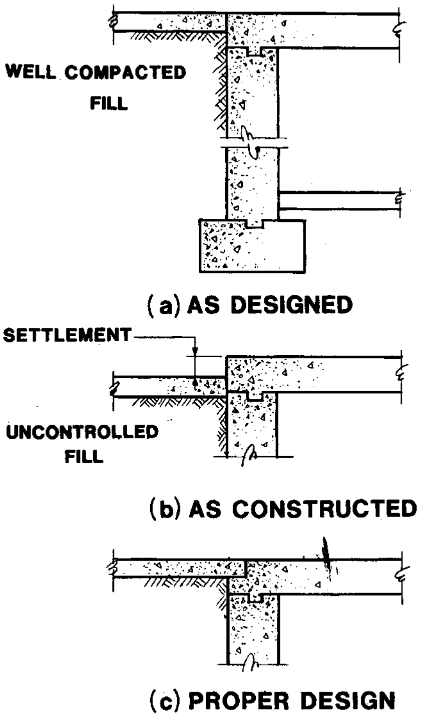


FIGURE 8.53 Slab design on fill.

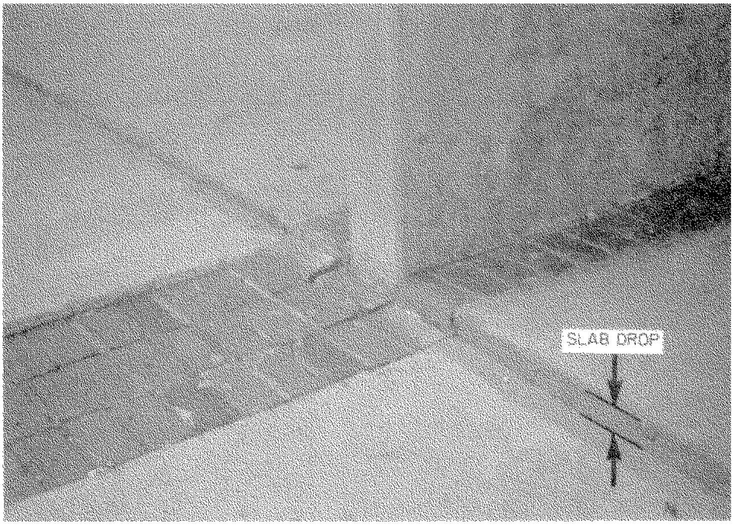


FIGURE 8.54 Slab settlement.

The unique cases involve the placement of foundation elements such as footings, piers, or piles on inadequate bearing soil strata—footings bearing on soft soils, or piles driven "short" so as to hang high in improper and insufficient bearing soil.

In 1958, a newly constructed 11-story building in Rio de Janeiro toppled and overturned. It was reported that the driven piles were about 20 ft (6.1 m) shorter than piles used for adjacent structures. Initially the settlement of the building was considered to be "natural." After several days of attempts to save the leaning building, the building was evacuated. Hours after the evacuation, the structure fell flat in *just 20 seconds.*

8.8.1 Lower East Side Collapses

On April 4, 1984, two structures located at Delancey Street in lower Manhattan collapsed during construction (Figure 8.55) about 1 h after a 3 ¼ in (89 mm) concrete floor slab had been poured. Two workers were killed by the collapse, including the general superintendent for construction.

On arrival on the scene a day later, I found that the debris was still being removed from the site. There was severe damage to the two adjoining structures, both of which were in precarious condition. One structure was subsequently demolished, and the other was braced and shored. Our investigation determined several improper construction procedures including inadequate field supervision. We also identified a number of design deficiencies.

Upon exposure after the collapse, several of the footings were found to have been cast as less than half (46%) the size required by the design drawings. Out of three interior footings, two footings had been cast eccentrically to the center line location [one 7½ in (191 mm) and the other by as much as 11½ in (292 mm)]. The eccentrically positioned footings were visibly rotated and pitched from one end to the other by as much as 5 in (127 mm). The general layout of the building may be seen in Figure 8.56.

The structural analysis indicated that the bearing capacity of these footings was considerably reduced. Using the actual loads computed to be present at the time of collapse, we estimated that even using the optimum possible bearing capacity of these footings, they would have been loaded at 25% over ultimate capacity and had to fail. See Figures 8.57 and 8.58 for footing diagrams and load table.

FIGURE 8.55 Collapse of structure.

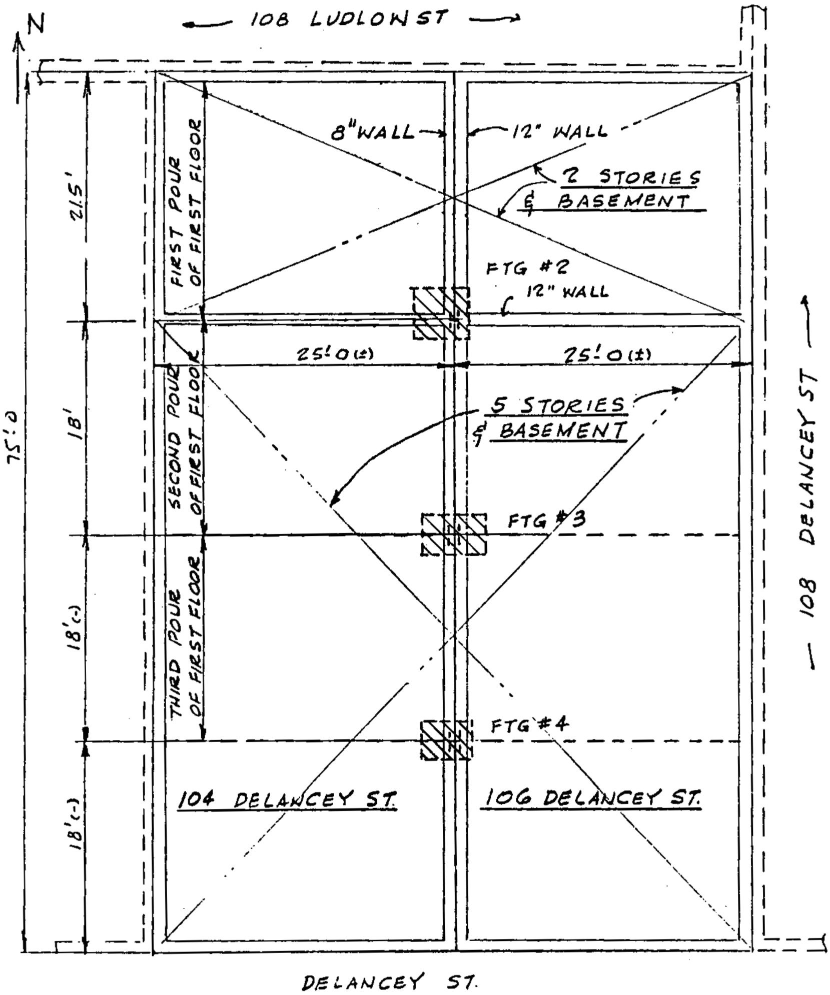


FIGURE 8.56 Floor plan of collapsed building (see Figure 8.55).

Another serious deficiency we uncovered related to the method of temporary support provided for the central bearing walls. These walls were subjected to a twisting rotational couple due to the weight of the two unequal bearing walls (Figure 8.59). The fact that the second floor was missing (it had previously been removed) compounded this effect through the lack of lateral support. The method of temporary support used by the contractor had a considerable inherent potential for instability. Thus, when the footings settled vertically, dislodging wedges and shims, the resultant movements initiated the total collapse.

One of the striking omissions in this failure was the lack of a construction procedure, which is essential to the successful execution of a difficult foundation reconstruction of an old structure.

The design deficiencies included main girders which were calculated to be stressed between 33 to 39 ksi (228 to 268 MPa) when subjected to full dead and live loads. Even the review of the design of the footings indicated overstresses in the range of 27 to 44%. Needless to say, the lack of shop drawings did not help matters.

FOOTING SIZE AND AREA			
FOOTING NO.	AS DESIGNED	AS CONSTRUCTED	REMARKS
FOOTING #2	5'-6"X 8'-6"=46.75sf	4'-8"X4'-7"= 21.4 sf	FOOTING CAST ECCENTRIC TO SUPPORTED COLUMN
FOOTING #3 (TOP AREA)	5'-6"X7'-0" = 38.5sf	5'-9"X3'-6" = 20.1 sf	FOOTING CONCENTRIC WITH SUPPORTED COLUMN BUT SIDE SLOPES, RESULTING IN AN ECCENTRIC BEARING SURFACE
FOOTING #3 (BOTTOM AREA)	5'-6"X7'-0"= 38.5sf	3'-9"X 3'-6" = 13.1sf	
FOOTING #4	5'-6"X7'-0" = 38.5sf	4'-2"X 3'-4"=13.9sf	FOOTING CAST ECCENTRIC TO SUPPORTED COLUMN

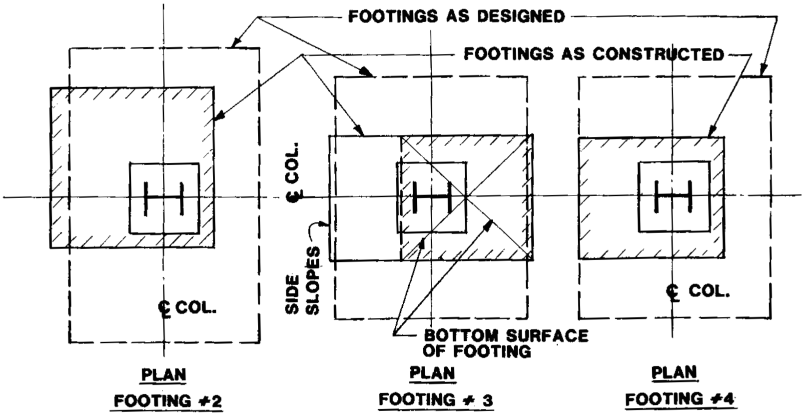


FIGURE 8.57 Substandard footings responsible for collapse (Figure 8.55).

Lessons to be learned:

1. Eccentrically loaded footings should be avoided at all costs, unless provided for in the design by oversized footings.
2. A construction procedure outlining step by step the method of temporary support, underpinning, shoring, and bracings is a must in construction of new foundations for old and deteriorated structures.
3. Lateral stability and eccentric effects on critical bearing walls must be considered and accounted for.
4. There is no substitute for good control of a project, including the preparation of shop drawings and thorough field inspection.

8.8.2 Apple Juice Factory—Drilled-in Piers

When I first saw this project, which had come to a halt, both owners and contractors were aware of the serious problem they were facing. The exposed sides of several concrete piers could be seen to contain concrete contaminated with large quantities of brown mud (Figure 8.60). Windsor probe tests had already been taken, and these confirmed the nonuniformity of the strength of the concrete. It was absolutely clear to everybody that expensive corrective work was required. The nature and extent of this work and the causes for the defects were sure to be in contention.

The foundation was being constructed to support an apple juice plant. The foundation system-

	1*	2*	3†	4†	5	6	7	8
	ALLOWABLE LOADS		DESIGN LOADS		DES.LD. ÷ ALLOW.LD.		DES.LD. ÷ ALLOW. LD.	
	N.Y.C. BLDG. CODE	ONE-THIRD OF ULTIMATE	WITH 1-12" & 1-8" WALLS	1-12" WALL ONLY	3 ÷ 1	3 ÷ 2	4 ÷ 1	4 ÷ 2
FOOTING #2	128^K	67^K	336^K	315^K	2.63 (163% OVER)	5.03 (403% OVER)	2.46 (146% OVER)	4.70 (370% OVER)
FOOTING #3 (FULL AREA)	121^K	216^K	358^K	313^K	2.95 (195% OVER)	1.66 (66% OVER)	2.59 (159% OVER)	1.45 (45% OVER)
FOOTING #3 (BOTTOM AREA)	79^K	272^K	358^K	313^K	4.53 (353% OVER)	1.32 (32% OVER)	3.96 (296% OVER)	1.15 (15% OVER)
FOOTING #4	83^K	114^K	358^K	313^K	4.30 (330% OVER)	3.14 (214% OVER)	3.77 (277% OVER)	2.74 (174% OVER)

	9	10	11**	12	13
	ESTIMATED LOAD AT TIME OF COLLAPSE		ULTIMATE LOAD	COLLAPSE LD. ÷ ULTIMATE LD.	
	WITH 1-12" & 1-8" WALLS	WITH 1-12" WALL ONLY	(PER WOODWARD-CLYDE)	9 ÷ 11	10 ÷ 11
FOOTING #2	251^K	217^K	200^K	1.26 (26% OVER)	1.09 (9.0% OVER)
FOOTING #3 (FULL AREA)	210^K	142^K	648^K	0.32 (32%)	0.22 (22%)
FOOTING #3 (BOTTOM AREA)	210^K	142^K	167^K	1.26 (26% OVER)	0.85 (85%)
FOOTING #4	210^K	142^K	342^K	0.61 (61%)	0.42 (42%)

ALLOWABLE BEARING DETERMINED BY WOODWARD-CLYDE CONSULTANTS INC.
*BASED ON ACTUAL FOOTING SIZES; N.Y.C. BLDG. CODE ALLOWABLE, 3 TONS PER SQ.FT.; EFFECT OF ECCENTRICITY NOT INCLUDED.
**EFFECT OF ECCENTRICITY CONSIDERED.
†DESIGN LOADS USED BY OUR FIRM.

	LIVE LOADS
1ST FLOOR:	100 PSF (RETAIL)
2ND FLOOR:	75 PSF (RETAIL)
3RD, 4TH, 5TH FLOORS:	100 PSF (LIGHT STORAGE)
ROOF:	30 PSF

FIGURE 8.58 Load table illustrates design flaws (see Figure 8.55).

consisted of concrete drilled-in piers (caissons), which were cast in place into big holes augured into the ground. These piers ranged in size from 30 to 42 in (762 to 1067 mm) diameter, with "bells" of 42 to 90 in (1067 to 2286 mm) diameter, and were 15 to 55 ft (5 to 17 m) long. The piers were designed to bear on top of bedrock with a bearing value of 12 tons per ft^2 (1149 kPa). Unfortunately, all the piers bad been completed at the time the defects were discovered.

The design of drilled piers with bells in clayey or silty sand is feasible provided the soil in which the bells are formed is cohesive. That condition may be realized where the bells are above the ground-water level. Where the shafts are flooded, as the case often was at this particular construction site, the success of belling is highly questionable. Especially doubtful is the possibility of installing the bells and maintaining them intact without the sides caving in and collapsing (Figure 8.61). The failed caissons confirmed this opinion. The presence of groundwater at several of the caissons should have been expected from the 23 borings taken at the site prior to construction.

What was more stunning was the interference of the design Engineer of Record with the contractor's methods and operations. When it became clear that it was impossible to pump the holes dry, the engineer directed the subcontractor to use larger pumps. [The ground-water infiltration was greater than the capacities of the pumps by more than 1000 gal/min (6308 L/s).] When the contractor did

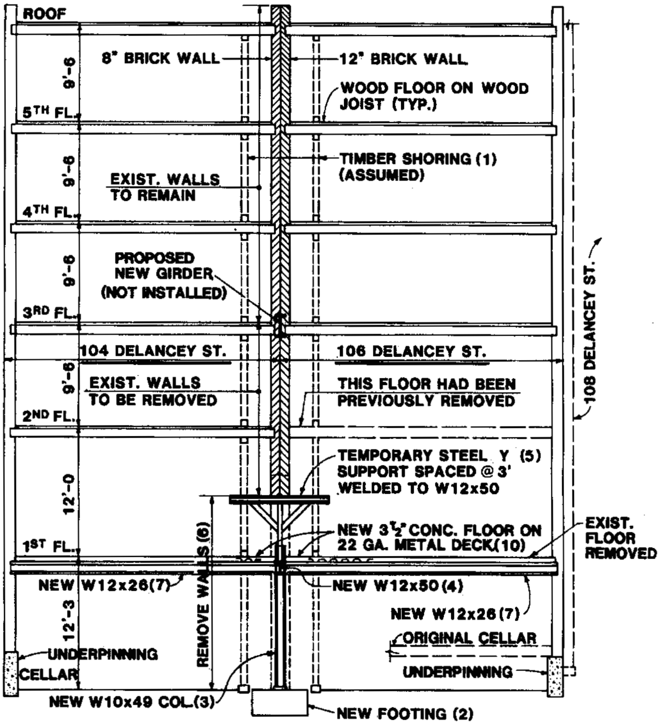


FIGURE 8.59 Construction alternatives contribute to collapse (see Figure 8.56).

not comply, the engineer bought large pumps using his own funds and delivered them to the contractor.

Thus the Engineer of Record committed three serious errors:

1. *A first major technical error.* It should be common knowledge that it is improper to cast concrete into holes while simultaneously pumping the holes. Such pumping only serves to disturb the wet concrete and mix it with soil, resulting in contaminated and defective concrete as per Figure 8.61 (properly filled caisson is shown in Figure 8.62).
2. *A second technical error.* There was no way for the engineer's inexperienced inspector to verify in the flooded shafts the conditions of the bearing rock and whether the rock was level or sloped (Figure 8.63).
3. *A policy error.* An engineer should never interfere with the contractor's methods and means, and definitely should not supply the contractor with the tools to perform the engineer's instructions, which also happened to be completely wrong. Needless to say, the subcontractor for the drilled piers should not have knowingly followed the erroneous instructions or should have known as a supposed expert in his field that they were flawed.

An experienced caisson contractor would also have known to continuously keep the bottom of the tremie (underwater concrete) pipe inserted into the fresh concrete (Figure 8.64). Whereas the selected foundation system for this structure was totally inappropriate (a system of H piles or even a continuous concrete mat should have been used), the construction methods used failed to produce a re-

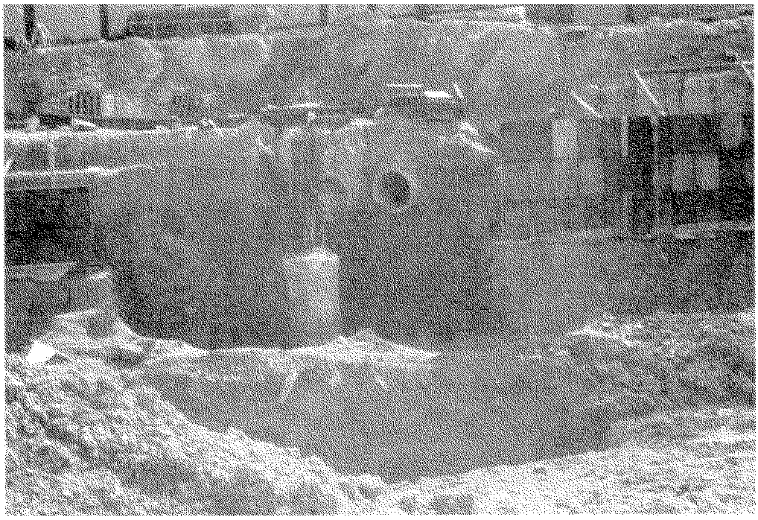


FIGURE 8.60 Defective concrete piers.

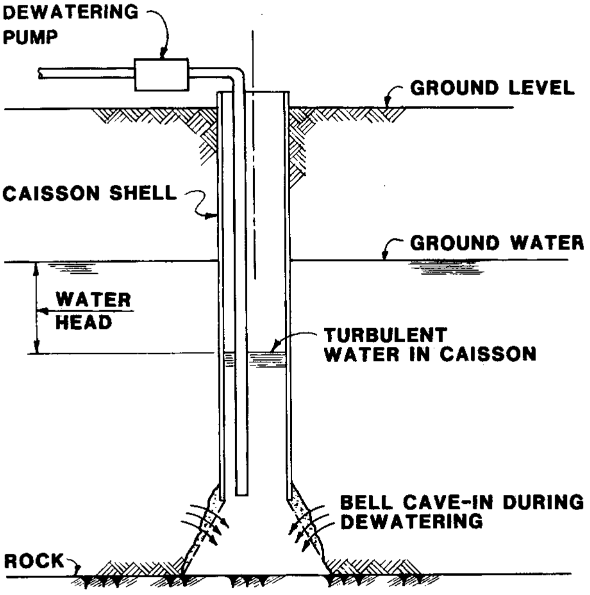
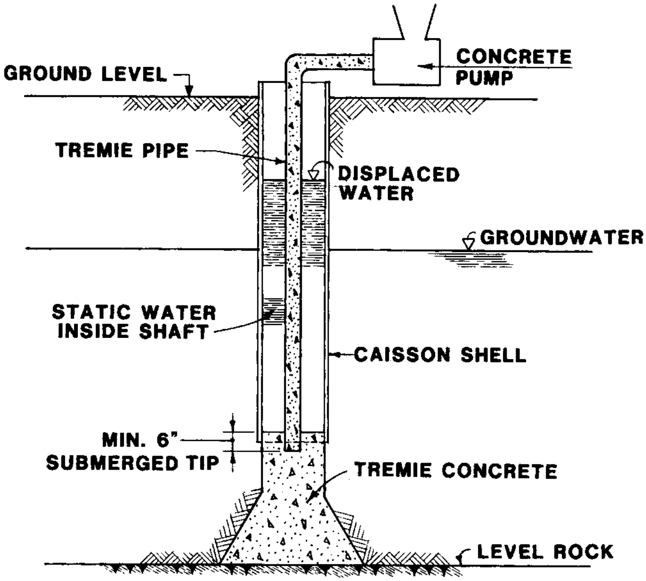
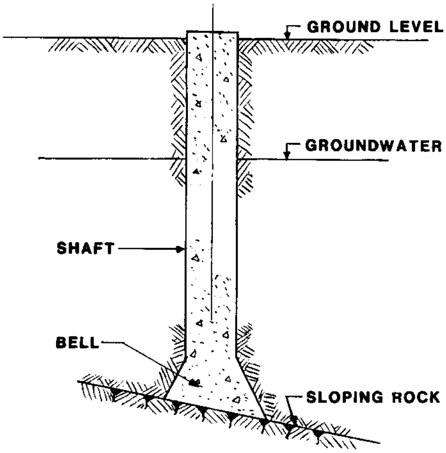


FIGURE 8.61 Caisson bell collapse.



CAISSON PROPERLY FILLED

FIGURE 8.62 Proper concrete placement in caisson.



IMPROPERLY SEATED CAISSON

FIGURE 8.63 Pier construction on sloping rock.

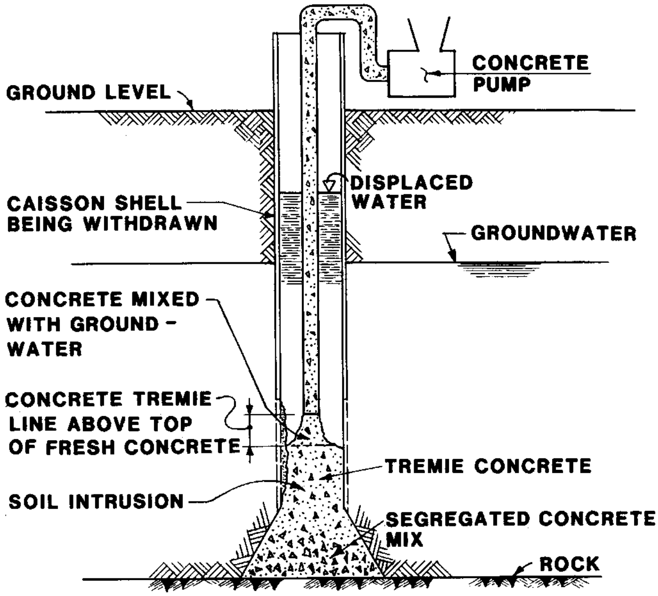


FIGURE 8.64 Improper use of tremie.

liable foundation. Not only was the concrete defective in many instances, but even the quality and levelness of the rock bearing strata could not be confirmed.

The sad ending to this story was that the foundation system as constructed had to be completely abandoned, and lengthy litigation ensued.

Lessons to be learned:

1. Designers should carefully check the feasibility of bell excavations in soil prior to concrete casting and especially in water.
2. Tremie placing of concrete should always be in still (static) water. Pumping during concrete placement operations should not be attempted.
3. The installation of drilled concrete piers should be performed only by experienced contractors.
4. Full-depth concrete test cores should be extracted from the first five piers, to verify the quality and strength of the concrete. The early detection of deficiencies is essential.
5. Engineers should not direct contractors' field operations, should not supply them with tools to perform the work, and should not interfere with their methods and means unless safety is involved.

8.8.3 Marble Hill

When cracks in the brick wall of the Marble Hill one-story structure started to widen, it became apparent that a serious condition existed below the concrete grade beam supporting the wall. After the soil adjacent to the grade beam was removed by excavation, a long diagonal crack was uncovered in the beam. The crack had been previously crudely patched with mortar, which itself was badly cracked and served no purpose whatsoever. There was no attempt to find the culprit! The failure was a classical shear crack with a vertical shift on both sides of the crack. A close look at this brick wall told the story of a settlement that had taken place prior to its construction. There was an additional course of brick on the left side of the crack (Figure 8.65). The solution to this mystery was found when, after further excavation, it was discovered that a pile designed to support the grade beam was missing. It was clearly a construction error. The attempt to patch the crack obviously was not adequate to correct the serious omission.

Lesson to be learned:

When cracking occurs, determine the cause of the distress. Do not guess! Only when the true causes are found can an efficient, long-term correction be designed and executed.

8.9 FLOTATION AND WATER-LEVEL CHANGE

Except in well-consolidated granular soils, a change in water content will modify the dimensions and structure of the supporting soil, whether from flooding or from dewatering. Many cases have been recorded where pumping by occupants for cooling water or by water companies to increase the capacity of their water supply resulted in receding of ground levels and in turn caused settlements with severe damage.

Pumping from adjacent construction excavations has also affected the stability of existing spread footings and even caused drag-down on short piles. This fact is in part responsible for the insertion of the requirement of recharging of ground-water levels in some foundation contracts. The effectiveness of these recharging pools has been and continues to be a touchy subject among foundation designers and geotechnical engineers.

Construction of new dams has also been found to be responsible for lowering of river levels, thereby causing severe damage and cracking of adjacent structures.

Clay heaves from oversaturation must also be expected and, in such soils, the structures must either be designed to tolerate upward displacement or else the supporting soil must be protected against flooding. Many parts of the world have trouble from heaving bentonitic clays, for which a

FIGURE 8.65 Foundation wall failure during construction.

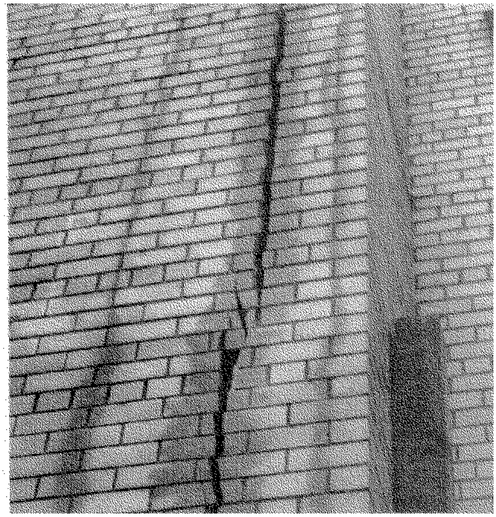


FIGURE 8.66 Vertical stress crack, hospital.

perfect solution is yet to be found. Where water infiltration into the soil is effectively prevented, and there is no change in water level, the soil volume should stay stable.

8.9.1 Brooklyn Hospital—Settlement of Delivery Room

This three-story hospital had performed perfectly for many years after its construction. Suddenly, masonry walls started to crack, "for no apparent reason" (Figure 8.66). The most severely affected wing of the building contained both the newborn delivery room and the general surgery room, whose continuous operation could not be stopped. I was actually measuring cracks inside the delivery room (dressed in scrubs) while a newborn was delivered. It was then that I found it necessary to temporarily support this wing, accomplished by the use of diagonal steel bracing system (Figure 8.67).

Once the structure was secured, we started the investigation of the sudden cracking and distress. To our amazement we found that the water supply company operating in the area had recently abandoned a great number of existing wells.

As a result, the ground-water level had risen sharply in the entire area. The immediate result was water penetration through foundation walls of the hospital which had not been waterproofed to such a high level. To solve the new problem of water infiltration into basement areas, pumps had then been installed by the hospital management. However, an unforeseen side effect was that the continuous pumping caused the removal of most of the fines in the soil directly below the footings supporting the affected wing, thereby causing the cracks and settlements.

There was no easy solution short of underpinning the building. This was accomplished by the use of jack-piles, which were pushed down into the ground, with the weight of the building serving as counterweights (Figure 8.68). The steel pipes which had been pretested for the required load were then filled with concrete, and then the jacks were removed.

Lessons to be learned:

1. Lowering or raising of ground-water level affects the bearing capacity of soils.
2. Pumping for new excavations may cause settlements of buildings. Therefore, water-level readings should be monitored, and protective measures taken.

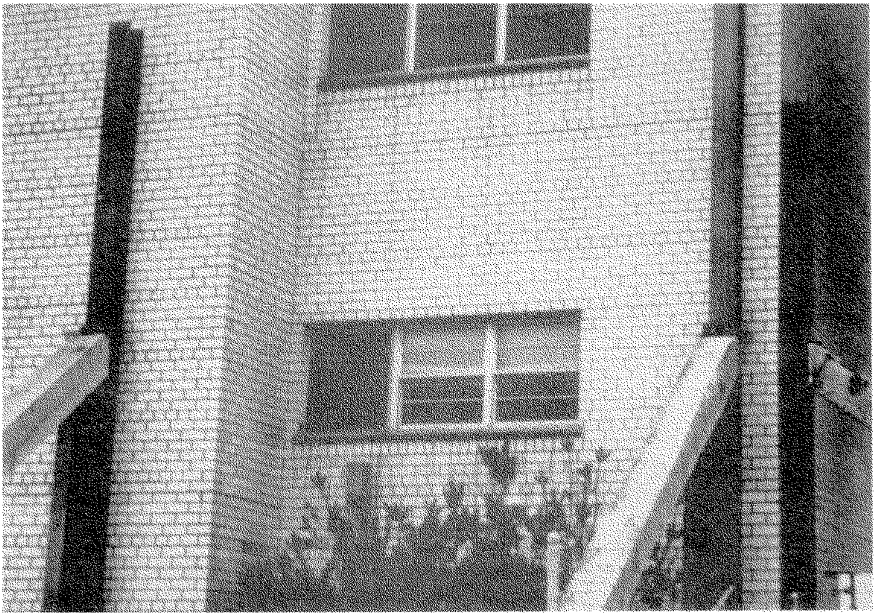


FIGURE 8.67 Shoring to abate settlement (see Figure 8.66).

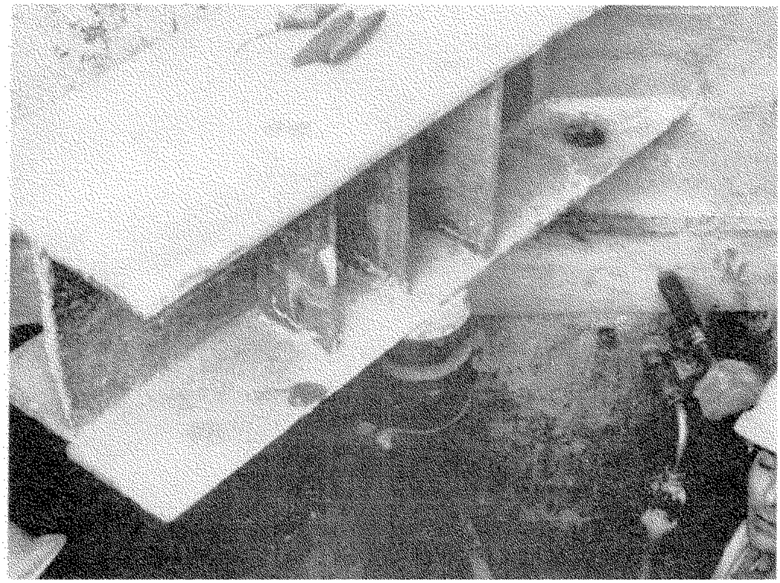


FIGURE 8.68 Underpinning and slabilizing building corner (see Figure 8.66).

8.9.2 Industrial Building, Hackensack, New Jersey

This one-story factory and office building was located in the bottom of a geological lake, atop deep glacial deposits of silts and clays in successive layers, forming what is known technically as varved silt deposit. In this very loose material, areas of soft shore land become marsh land. Considerable areas of such marsh land have been filled in to form usable development property. The existence of the soft underlying material should signal potential difficulties and must be taken into account in all construction operations. This type of soil is very susceptible to drainage and is associated with a considerable volume change.

The local sewer authority had issued a contract for the construction of a new sewer. A deep trench was excavated using steel "soldier beams" 10 ft (3.0 m) on center and timber sheeting. The excavation was braced at two levels. As the excavation approached the factory, various signs of damage started to appear in the form of cave-ins and vertical and diagonal cracking of walls and partitions (Figures 8.69 and 8.70).

The damage was reviewed by our firm, and it was evident that the undermining was caused by the drainage operations in the sewer trench causing a shrinkage of the subsoil under the building's foundation. This was followed by the lateral movement of soil from under the building toward the deep hole at the sewer trench. The nearest edge of the sewer excavation was approximately 220 ft (67 m) from the building. Yet the effect of the drainage of the sewer excavation extended to great distances on each side of the sewer, and soil settlement was even visible in the parking area in addition to along the walls of the building itself.

Fortunately, the main structure of the building was supported on piles which had been carried

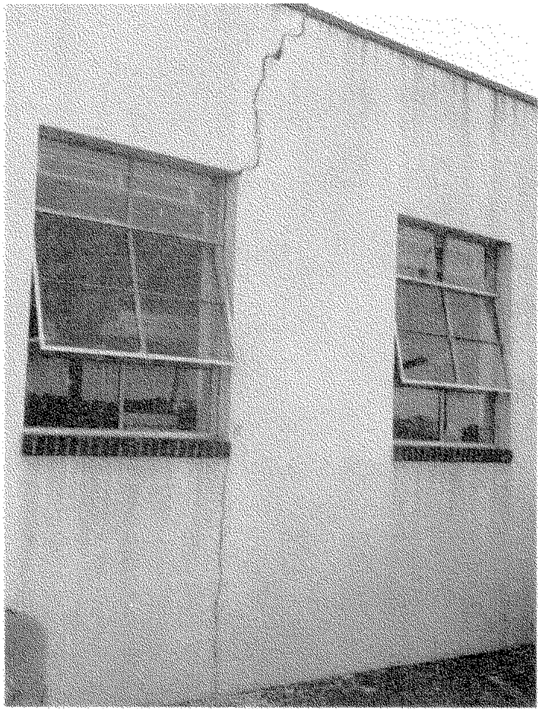


FIGURE 8.69 Masonry distress caused by soil shrinkage (loss of bearing support).

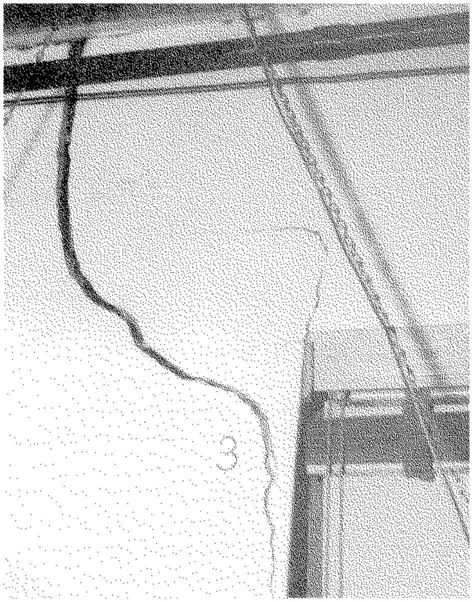


FIGURE 8.70 Interior distress (see Figure 8.69).

into materials deep enough and dense enough not to be affected by this drainage. There was, however, still the effect of "negative friction" on the piles, with the additional soil weight adding to the loads which were already present.

Once the pumping operations ceased, no additional damage was recorded. Slabs which had settled by as much as 3 to 4 in (76 to 101 mm) had to be removed and rebuilt. Slabs which settled to a lesser degree, leaving a hollow space between the underside of the slab and the ground, were restored by pressure grouting. Wall cracks were repaired using ordinary cement mortar and refinished.

Lessons to be learned.

1. The ground-water level outside a deep excavation should be monitored using piezometer pipes installed at the adjacent structures which may be affected. Criteria for the maximum permissible water-level drop should be set.
2. In the event of a drop exceeding the criteria and/or the initiation of damage, the pumping methods and procedures should be reviewed to minimize the damage.
3. The services of competent geotechnical and structural engineers should be solicited.

8.9.3 West 41st Street Structure—Wood Piles

Damage to the exterior brick masonry of this building was caused by settlement of timber piles resulting from lowering of ground-water level at an adjacent railroad construction. Probes below the building showed that the timber piles were rotted and could not accommodate the additional loads generated by the drop of the water level. The damage to exterior and interior masonry is shown in Figures 8.71 and 8.72.

8.10 VIBRATION EFFECTS

Earth masses which are not fully consolidated will change volume when exposed to vibration impulses. The vibration source can be construction equipment, especially pile drivers, mechanical

FIGURE 8.71 Interior distress (Figure 8.71).

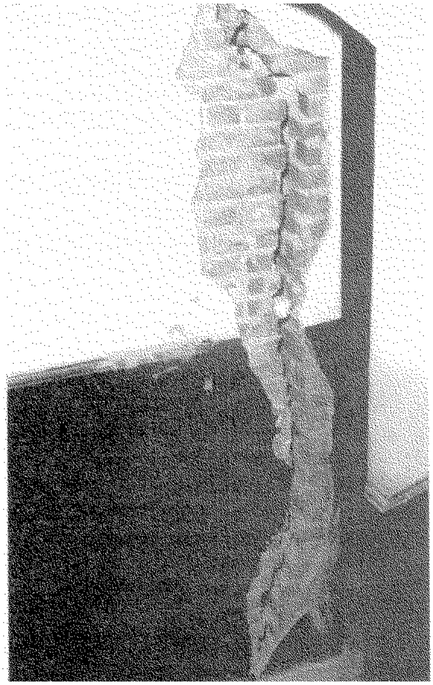


FIGURE 8.72 Interior distress (see Figure 8.71).

equipment in a completed building, or even traffic on rough or potholed pavement, as well as blasting shock.

We have investigated many cases of damage as a result of pile driving. Heavy damage can be caused when both impact hammers and vibratory hammers are used. In several cases, entire rows of buildings have had to be condemned as unsafe and were subsequently demolished.

Blasting operations must be carefully programmed, to avoid serious damage to adjacent structures. A criterion for maximum particle velocity, which is a measure of the intensity of vibration, has to be established, and constant readings by the use of seismographs have to be taken. Damage from blasting is often so potentially severe that the size of explosive charges have to be limited to keep the vibrations to tolerable levels. Factors such as the condition, rigidity, material brittleness, and age of nearby structures must be considered. Where adjacent structures are founded on rock, the seam structure of the rock itself also must be taken into account. A complete study of vibration transmission and correlation among intensity, wave length, and possible damage to various types of structures is given in a Liberty Mutual Insurance Company report, "Ground Vibration Due to Blasting," by F. J. Crandell.[4]

8.10.1 Lower Manhattan Federal Office Building

After heavy impact hammers were used to install the piles for a prestigious building, the lower Manhattan Federal Office Building, the adjacent structures were seriously shaken. Street pavements settled (Figure 8.73) as much as 16 in (406 mm) after only one-fourth of the piles had been driven. The installation of steel sheetpiling was then attempted in an effort to protect the streets against further

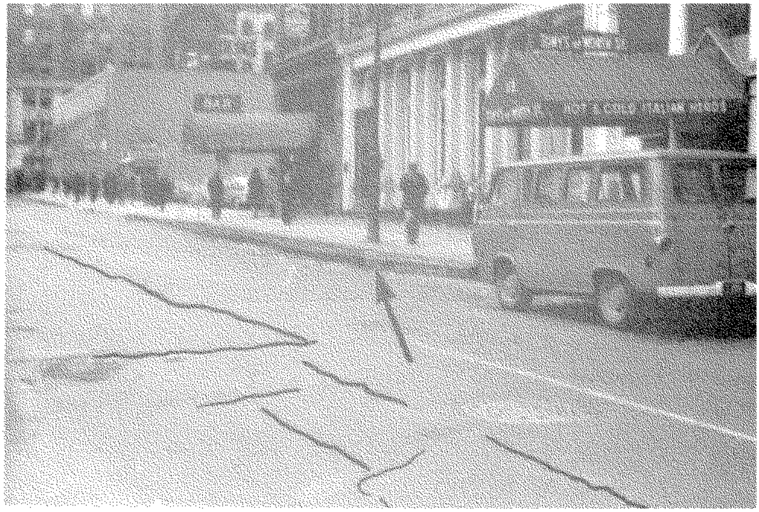


FIGURE 8.73 Pavement settlement due to pile-driving vibratins.

subsidence. This attempt was successful against further settlement and damage to sewer and water lines, but even after the contractor shifted to vibratory hammers, the damage continued. High-rise buildings as tall as 19 stories were also damaged. The damage was so heavy that eventually all buildings within a radius of 400 ft (122 m) were condemned as unsafe and had to be demolished. In a similar construction some 1000 ft (305 m) away, when first signs of trouble became evident, the adjacent buildings were underpinned on piles down to the tip level of the new building piles. From here on, the consolidation of sand layers as a result of pile driving had no ill effect, as these sand layers were no longer supporting building loads.

Lessons to be learned:

1. The use of vibratory pile-driving equipment must be avoided where possible. Such equipment should be used only after careful evaluation of potential damage.
2. Underpinning of adjacent structures must be seriously considered for support to minimize damage.
3. Where possible the use of augured piles should be considered.

8.11 COFFERDAMS—THE 14TH STREET COFFERDAM

In order to construct a deep pumping station picking up flow from a 9 ft (2.74 m) sewer on the east side of New York City, a temporary steel cofferdam was constructed. The size of the pit was 100 ft × 250 ft × 37 ft (30.48 in × 76.20 in × 11.28 m). Adjacent was a high-rise public housing building on short piles so that the excavation was 30 ft (9.14 m) below the pile tips.

The site was filled-in land overlying some organic silt and with sand 80 ft (24.38 m) down, showing full hydrostatic pressure. It was not possible to drive interlocking sheetpiling through the sand to impervious till or rock, about 95 ft (28.96 m), since major removal of the rough fill might affect the stability of the apartment building. The solution was to core through the fills and place 30 in (762 mm) pipe sleeves and drive soldier beams on 5 ft (1.52 m) spacing, first welding sheetpile interlocks to two edges of the wide flange (see Figure 8.74). The soldiers were driven to bedrock.

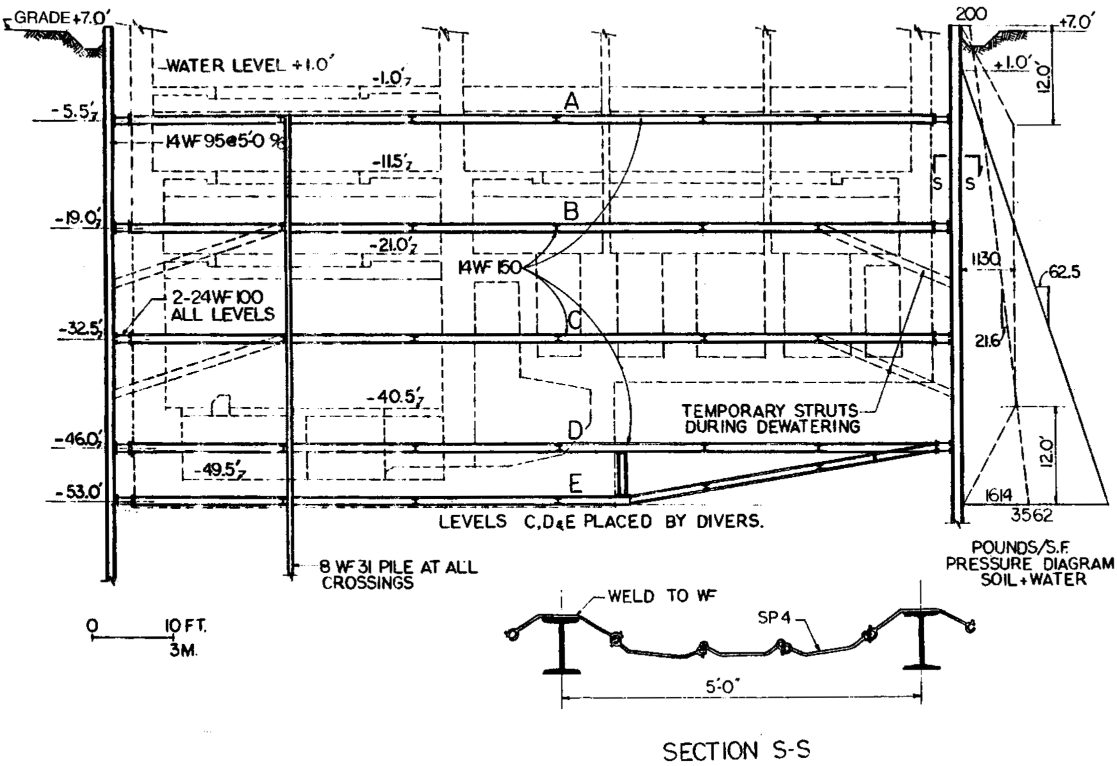


FIGURE 8.74 Soldier beams used to construct cofferdam.

The space between the soldiers was then filled in with three units of flat steel sheeting, draped into a catenary arc and designed to act as such. Sheeting was driven into the till layer. The horizontal cross-bracings were all installed by jacking design loads into all members in each direction. Bracing levels were installed as the cofferdam was excavated and pumped out. Water was controlled by two deep wells carried into the sand. Some of the sheeting engaged an old buried timber crib, and some of the interlocks were broken so that water cutoff was not accomplished.

Although the basic design of the interlocking sheet piles was proper, neither the 5 ft (1.52 m) spacing nor the verticality of the piles could be accurately controlled. Thus, the cofferdam walls failed at certain locations as the sheet-pile fingers tore, leaving the wall with open gaps for inflow of soil.

After several blows through the bent sheeting stopped progress, a slurry trench cutoff to rock was installed along the distorted and damaged section, which fortunately was located furthest from the apartment building. The open area near the apartment building was recharged with pumped water to maintain the original water table level, and no damage resulted either to the building or to the trees.

Lesson to be learned:

Driving of sheetpiling should be closely controlled so that extremely hard driving with potential damage to the sheets can be avoided.

8.12 CAISSONS AND PILES

8.12.1 The North River Project—Caisson Deficiencies

Manhattan's enormous North River water pollution project was constructed on a 30-acre (121,400 m^2) concrete platform supported on 2500 drilled caissons (Figure 8.75). The caissons extend to the Hudson River bottom and are socketed into the rock underlying the lower sand layers. They were from 80 to 250 ft (24 to 76 m) long and were constructed of steel pipe shells in diameters varying from 36 to 42 in (914 to 1067 mm).

For economy reasons, the caissons were designed to carry very heavy loads of up to a maximum of 5000 tons (45 MN) each. During the construction of the platform, two major problems arose:

1. Defective concrete was found to be cast in the caisson shells.
2. Caissons shifted laterally at the top and bent. These problems came to light after a substantial number of the caissons had been completed.

A full-scale study was launched, and it was determined that the problem of the defective concrete was caused by improper tremie concrete placement. It became clear early during the installation of the caissons that about one-quarter of the caissons had low-strength concrete and discontinuities of different varieties: uncemented aggregates, unhydrated cement, and sand layers intermixed with the concrete.

After driving the steel shells into the river, the shells were seated into the rock. Sockets were then drilled in the rock, and the shells were emptied of river mud and sand. The contractor made several attempts at casting the caissons in the dry cement after sealing the bottom and pumping the water out of the shell. When these attempts failed, a tremie method (casting concrete in water) was used. This method, which unfortunately did not utilize pressure pumping of concrete, also failed, a fact that was brought to light only later. This was so because the accidental lateral shift of the caissons

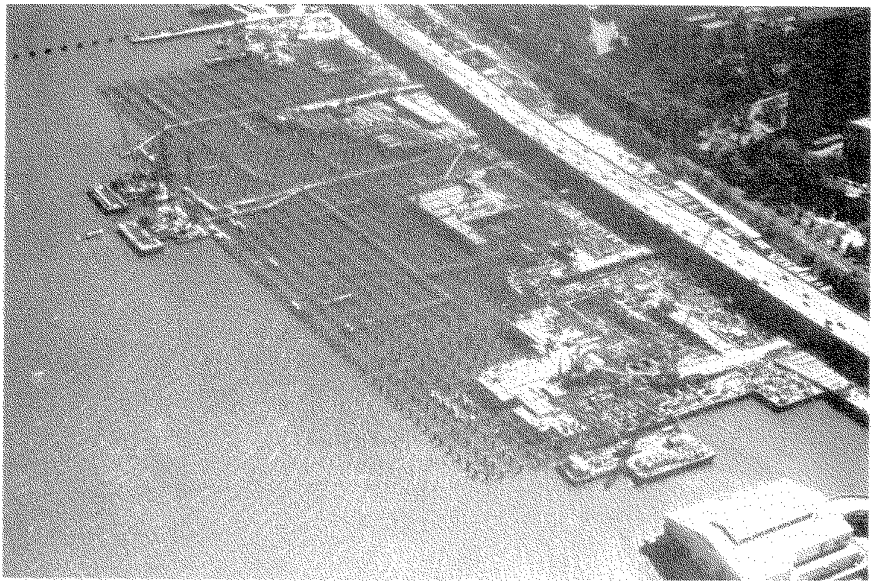


FIGURE 8.75 2500 caissons installed in Hudson River.

caused sufficient concern, necessitating full-scale in-place load tests. The first of the load tests (described later) resulted in failure of the tested caisson, which sank vertically, a failure which confirmed the concrete defects within the caissons.

The concrete subsequently was repaired by new high-strength rebar [75 ksi (517 MPa)] bundles installed in vertical holes drilled in the caissons. These holes were then pressure-grouted with high-strength grout.

The *shifting of the tops* was determined to be the result of one-sided soil dredging of a deep trench. The creation of the trench caused unbalanced lateral soil pressure on the caissons, resulting in horizontal movement and excessive bowing. This horizontal movement of the tops exceeded 2 ft (610 mm) in many cases. The caissons were later forced back into position by a "pulling program" involving the application of high lateral loads coupled with jetting of the caissons. It was only after the lateral shift of several hundred caissons occurred that the concrete problem was discovered, as it was decided to core-drill several of the caissons. The coring revealed many deficiencies within the caissons, where some "concrete" near the bottom of the caissons was nothing but sand (Figure 8.76). Before any remedial measures were considered and proposed, it was decided to load-test one of the affected caissons. To determine the load capacity of the shifted and bent caissons, a full-scale load test was ordered. The tested caisson was 36 in (914mm) in diameter, 98 ft (30 m) long, socketed 5 ft (1.5 m) into rock, with a design capacity of 680 tons (6.1 MN). The top of this caisson had been displaced laterally as much as 2.35 ft (0.71 m) after it was installed.

The vertical load was applied to the test caisson by means of four hydraulic jacks; the jacks reacted against a loading frame held in place by four vertical cable anchorages grouted 45 ft (13.7 m) into sound rock (Figure 8.77).

Four 0.001 in (0.025 mm) dial extensometer gages were mounted on an independent common frame to measure vertical displacements at the top of the caisson. The top of the test caisson was braced at the top to adjacent caissons to prevent lateral translation during the test. The braces were pin-connected to eliminate undesired transferral of the vertical load through the bracing system. The load was applied in two phases.

During the first phase, a load of 850 tons (8.5 MN) was applied. The load was cycled in small increments. All loads were maintained until the vertical displacements were stabilized. Only the 765- and 850-ton (6.8- and 7.5-MN) loads were maintained for 15 and 18 h, respectively. Later the load was *cycled five times.* The vertical displacement under a load of 850 tons (7.5 MN) (125% of the design load) after 17 h was 2.6 in (66 mm) with a permanent set of 2.2 in (56 mm).

During the second phase, the load was increased and cycled further in increments to a maximum of 1360 tons (12 MN).

This maximum load was held for 60 hours and then recycled again ten times. To everybody's shock, the caisson failed to support the load and sank. An additional deflection of 3.6 in (92 mm) was measured for a total vertical displacement of 5.8 in (147 mm). The total permanent set was 4.5 in (114 mm) (Figure 8.78).

The excessive vertical displacement and permanent set were caused by either one or a combination of two factors:

1. The existence of uncemented zones within the lower 40 ft (12.2 m) of the caisson resulted in the transfer of the entire caisson load to the steel shell which was subjected to a stress of 50 ksi (345 MPa).
2. Lacking sound concrete in the socket itself (there was actually uncemented sand in the socket), the cutting edge of the shell was punched further into the socket.

   It was not possible to determine whether the lateral shift of the caisson was, in fact, a significant factor causing the vertical displacement of the caisson and its eventual failure. For this reason an additional caisson was tested 2 months later. This later test proved that the lateral displacement of 2.5 ft (0.76 m) at the top of the caisson had no discernible effect on the elastic behavior and load-carrying capacity of the structurally sound portion of the caisson. Therefore, all caissons with displacements of less than 2.5 ft (0.76 m) were incorporated into the structure. The unbalanced lateral loads resulting from the lateral shift were subsequently balanced by batter caissons.

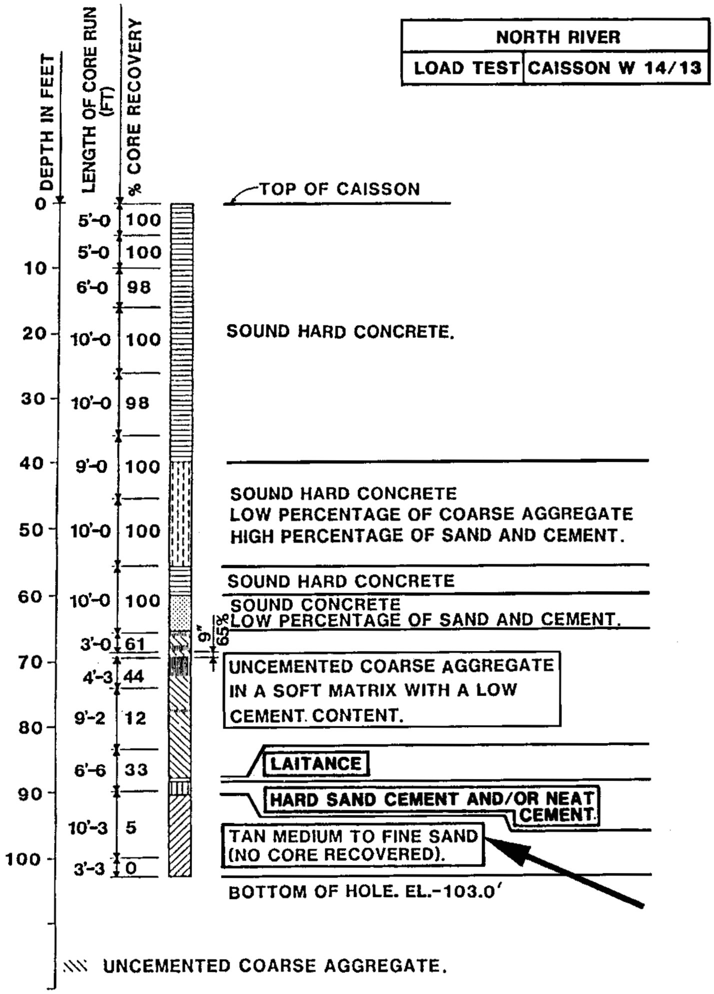


FIGURE 8.76 Coring log, load test caisson (see Figure 8.75).

A second defective caisson was then tested similarly. This caisson was also cored with an NX-size double tube core barrel prior to load testing. What we found was that the upper 80 ft (25 m) contained generally sound concrete, contrary to the first tested caisson. To measure the vertical displacement at the bottom of the caisson, measuring devices were inserted into 1 in (25 mm) diameter pipes installed in holes drilled in the caisson. The devices were ¼ in (13 mm) diameter smooth steel rod tell-tales lowered into the pipes. This caisson held the load for the required 60 hours, with a total displacement of only 0.35 in (9 mm). This led us to conclude that the caissons were repairable and could be incorporated into the structure without any reduction of their load capacity. We also determined that the presence of the powdery cement layer was apparently the result of the concreting procedures employed, whereby up to three bags of cement were deposited to absorb the water which was not removed by pumping out of the shell.

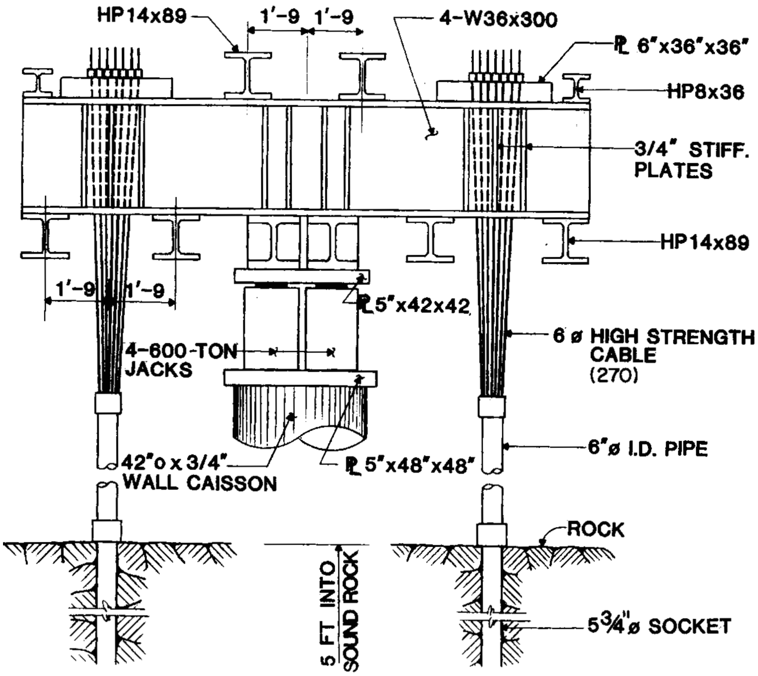


FIGURE 8.77 Test-loading questionable caissons (see Figure 8.76).

Remedial Work

The caissons were repaired by bridging the discontinuities with grouted rebars. Two 10 in (254 mm) holes were drilled within the concrete for the full depth of the caissons, including the socket, into rock. After each hole was cleaned of loose concrete, rebar bundles consisting of twelve no. 11 high-strength [75 ksi yield (517 MPa)] bars tied together around pipe spacers were lowered into the holes and grouted (Figures 8.79 and 8.80).

Lateral Shifting of the Top of the Caissons

As soon as it was discovered that a large group of caissons had shifted laterally after installation, an immediate investigation was launched. Survey teams using laser-based instruments took readings of 180 individual caissons to establish their accurate locations. The top of the caissons were surveyed for a year. (See typical movement chart in Figure 8.81). Lateral displacements of as much as 2.5 ft (0.76 m) were measured. Temperature variations were found (by analytical computation) to account for only a small portion of the movement. Examination of the horizontal steel 8HP36 staylath members (Figure 8.82) and their connections showed them to be in good condition and without any signs of distress. Analytical computations established that very high loads were, indeed, required to be generated to cause such a shift.

It was also determined that a one-sided soil pressure is capable of generating such high loads. The actual behavior of the group of caissons which were tied together by means of steel bracings followed the pattern of lateral shift resulting from an unbalanced lateral load. It was soon discovered that such unbalance did, in fact, occur as a result of a sizable dredging operation performed to remove existing boulders which were obstructing caisson-driving operations. It was established that, although most caissons moved west (toward the dredged trench, which was excavated parallel to the

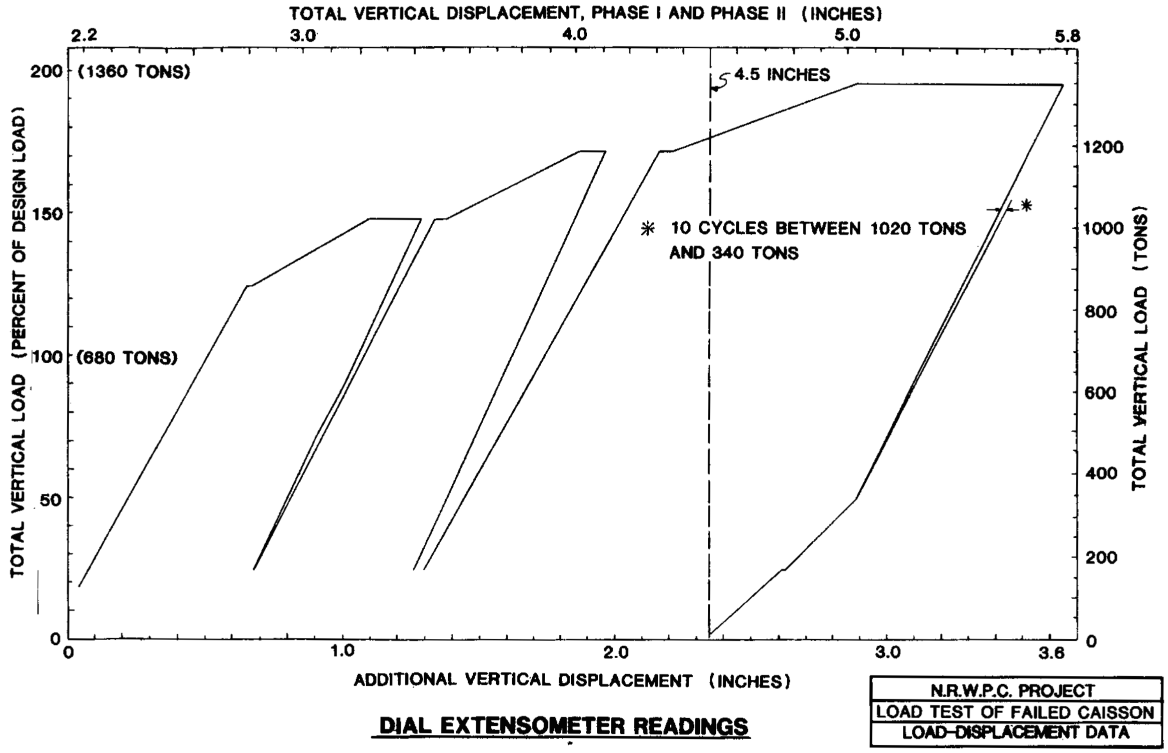


FIGURE 8.78 Vertical load versus vertical displacement for caissons.

shoreline), several caissons located west of the trench did, indeed, move toward the trench in the easterly direction.

At this time we had to determine whether the movements in the southeast section of the foundation systems were continuing and/or could be expected to continue after the completion of the concrete deck supported by the caissons. We also needed to assess the effect of such possible movements on the superstructure.

To answer these questions, we released a number of caissons from their staylath bracings and installed inclinometers in drilled holes within the caissons. The inclinometers, which are devices that measure tilt, or the angular deviation from the vertical, were the slope indicator type.

The results of the surveys (Figure 8.83) indicated that, except for the initial adjustment of the casing after installation, no movement was occurring. We therefore concluded that the westward movements of the caissons and/or soil in the southeast area of the project had ceased.

The main concern at this juncture was to establish whether the structural integrity of the caissons was still preserved. The stress analysis that followed did confirm this fact, and we then looked for a method of moving the displaced caissons back to their original vertical positions.

"Pulling" procedures were developed, using "come-alongs" to move the caissons laterally. The loads (measured by dynamometers) were applied in small increments, so as not to damage the caissons. Jetting of the river bottom was utilized to assist in pulling the caissons to the desired locations. It should be pointed out that the enormous construction difficulties encountered in this project were effectively resolved due to the excellent cooperation and combined efforts of the owner and both the design and construction teams.

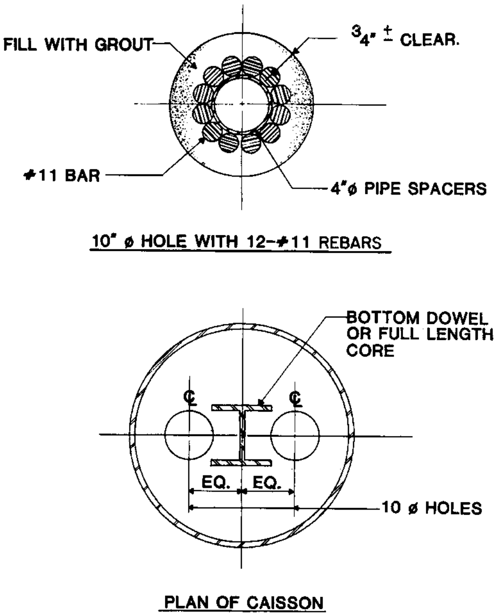


FIGURE 8.79 Repair of defective caissons.

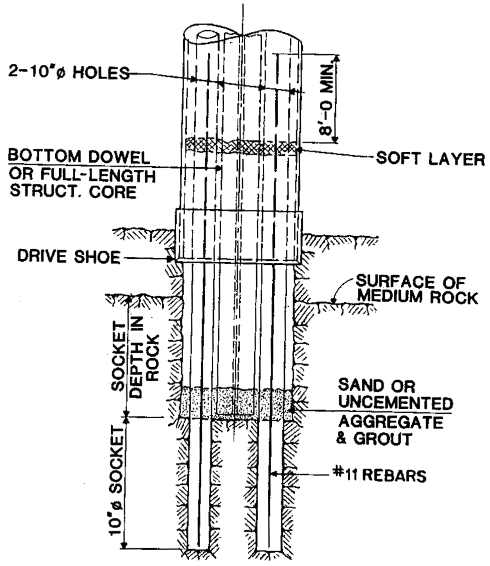


FIGURE 8.80 Schematic diagram for grouted rebar extended 12 ft (3.7 m) mb rock.

Lessons to be learned:

1. Tremie casting of concrete into long caissons should be performed under pressure by the use of pumps.
2. The bottom of the tremie pipe must at all times be embedded into the fresh concrete.
3. The quality and strength of concrete in caissons, and the reliability of the tremie concrete procedures, must be confirmed by core drilling at the beginning of the operation.
4. Cutting deep trenches adjacent to caissons or similar pile foundations will result in a lateral shift and should be avoided.
5. Good cooperation among the various parties involved in a construction failure is essential to a speedy, successful, and litigation-free resolution.

8.12.2 Philadelphia Federal Office Building—Caisson Settlement

In 1974, several piers of this 23-story steel frame high-rise building settled with one caisson settling approximately 2 in (50.8 mm). The base of the building was 274 ft × 240 ft (83.52 m × 73.15 m) with a narrow tower approximately 120 ft × 150 ft (36.58 m × 45.72 m) in size. Wind frames were constructed in two perpendicular directions by the use of welded and high-strength bolt connections.

After stress analysis was performed, it was decided to jack one of those columns that settled; however, no action was taken from 1974 to 1976 (2 years).

In 1977, four 400-ton (3559-kN) jacks were placed between the structural steel column and the caisson. The structure was to be raised 1 in (25 m) in two stages. The relative movement of the col-

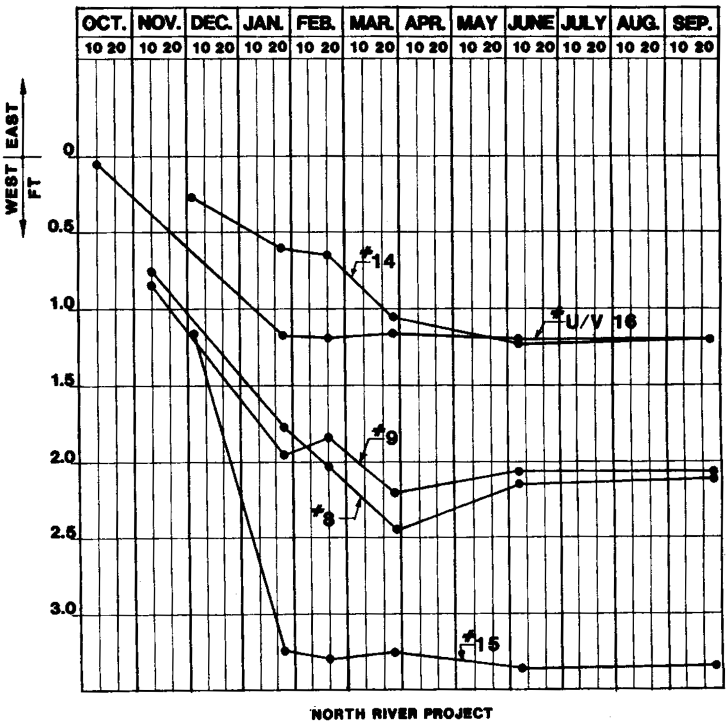


FIGURE 8.81 Test survey of caisson movement before and after stabilization.

umn in respect to the caisson was measured by dial gages. At this time accurate survey revealed that 44 caissons had settled.

The original plans called for the caissons to bear on rock at an approximate depth of 60 to 70 ft (18.29 in to 21.34 m), but later (1969) a change order was issued (with a $70,000 savings) which required the caisson to go down only 50 to 55 ft (15.24 in to 16.76 m).

During construction, there were reports of sudden drop of the top of the caisson and of water accumulation at the top of the caissons. There were two possible causes for the settlements of the caissons:

1. Inadequate bearing strata at the bottom of the caisson
2. Discontinuities and voids within the concrete caisson itself

The load test that was conducted was inconclusive, since the monitoring setup was not properly instrumented. Whereas only the top of the caisson was monitored, it was not possible to determine whether the bottom of the caisson dropped.

What was confirmed was that the jacking of the caisson actually resulted in a drop of the caisson instead of the lifting of the column above.

The final repair utilized grout which was injected into, around, and below the settled caissons.

Lessons to be learned:

1. Caisson construction must include coring of typical caissons to confirm the quality and continuity of the concrete. Coring program must be performed at the start of construction.

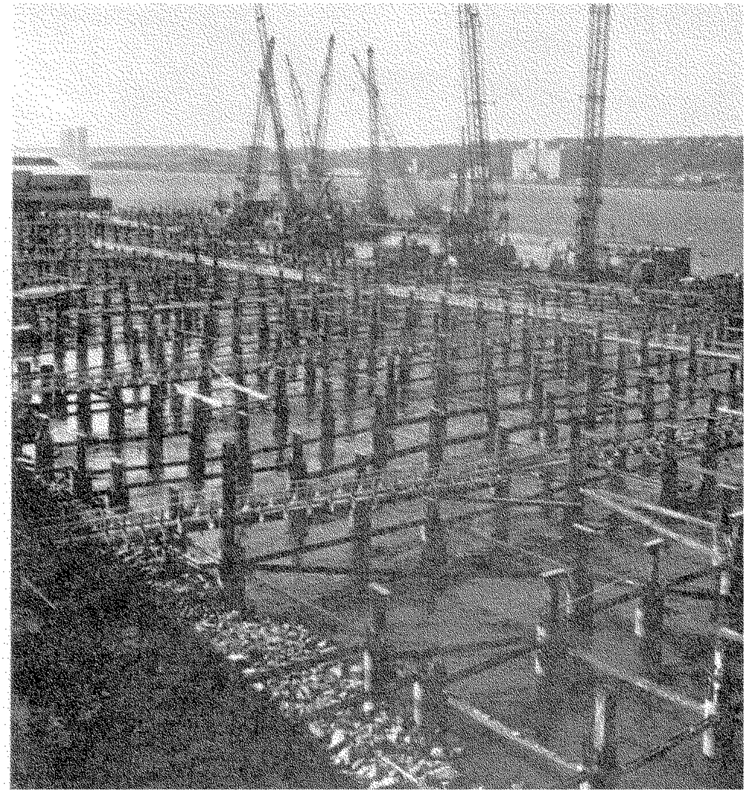


FIGURE 7.82 Bracing caissons (see Figure 8.75)

2. Bearing surface must be confirmed by 5 ft long (1.52 m) rock cores extracted prior to casting of the concrete.

Recommendations for design and construction improvement include avoiding common pitfalls and review of checklists.

8.13 COMMON PITFALLS

1. One structure supported on two different foundation systems, i.e., piles and spread footings
2. Inadequate drainage for retaining and basement walls, leading to buildup of hydrostatic pressure
3. Inadequate lateral stability of structures with one-sided soil pressure
4. Corrosion of piles and sheet piles in the splash zone
5. Lack of waterstops in foundation walls and pressure slabs

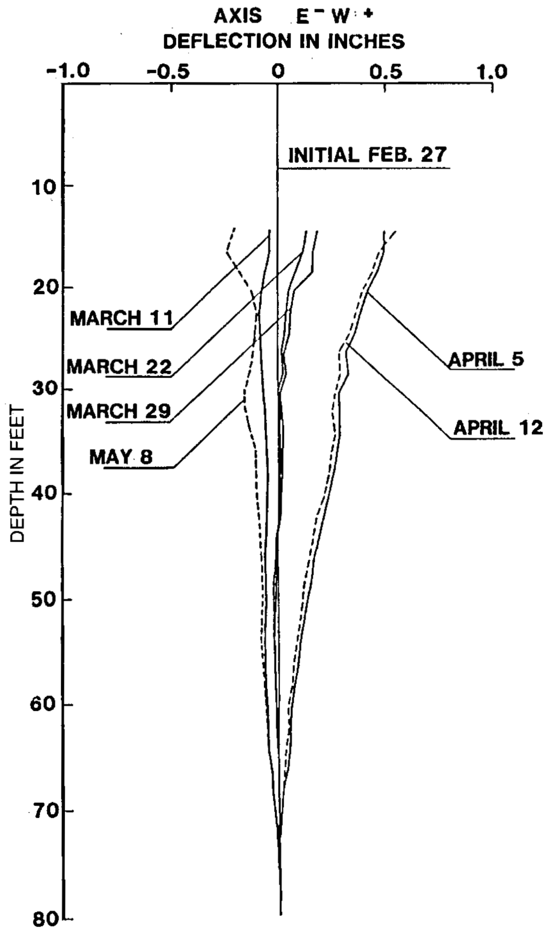


FIGURE 8.83 Inclinometer readings of caissons.

6. Ignoring negative friction effect on piles in unconsolidated soils
7. Lack of piles to support slabs on grade where soils are of poor bearing capacity
8. Insufficient or no borings
9. Borings not taken deep enough
10. No consideration of ground-water level
11. Failure to design for uniform settlement
12. Backfilling before foundation walls are adequately braced by floor framing
13. Inadequate depth of foundations subjected to scouring
14. Compacting upper 10 in (25.4 mm) layer of loose, unconsolidated soil and fills

8.14 CHECKLIST

1. Visit site to observe existing structures which may be affected by new construction. Find out type of foundation used for nearby existing buildings.
2. Take proper borings of sufficient number and adequate depth. Verify ground-water table.
3. Choose appropriate type of foundation. Consult geotechnical engineer.
4. Design for uniform bearing and settlement or provide for any differential settlements which may occur between parts of the structure, by use of expansion joints.
5. Brace foundation walls before backfilling. Do not load unbraced foundations with material or equipment surcharge.
6. Provide drainage at retaining or basement walls.
8. Verify that actual soil is as assumed in design, both for vertical support and for lateral load against walls below grade.
8. Check for effect that pumping and/or vibratory equipment may have on existing structures.
9. Design foundation for all directional loads and within allowable stresses and settlement tolerances.
10. Support slabs on piles when slabs are bearing on compressible or otherwise inadequate soils.
11. Provide step-by-step construction procedure when temporary support, underpinning, shoring, or bracing is required during installation of new foundation.
12. Tremie placing of concrete should always be performed in still water. Do not pump water out of the shell during placement operations.
13. For drilled pier and caisson installations, extract full-depth concrete test cores from the first five to verify integrity of the concrete and adequacy of bottom bearing.
14. Monitor ground-water level during construction, especially if pumping is required, since it may affect existing buildings and the bearing capacity for new structures.
15. Establish vibration criteria for effect on existing structures during pile-driving operations. Special care is required for old and historical landmark buildings.
16. Provide batter piles to resist lateral loads.
17. Avoid bending moments and eccentricities on footings.
18. Check pile caps for eccentricities of as-driven pile locations.

8.15 REFERENCES

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