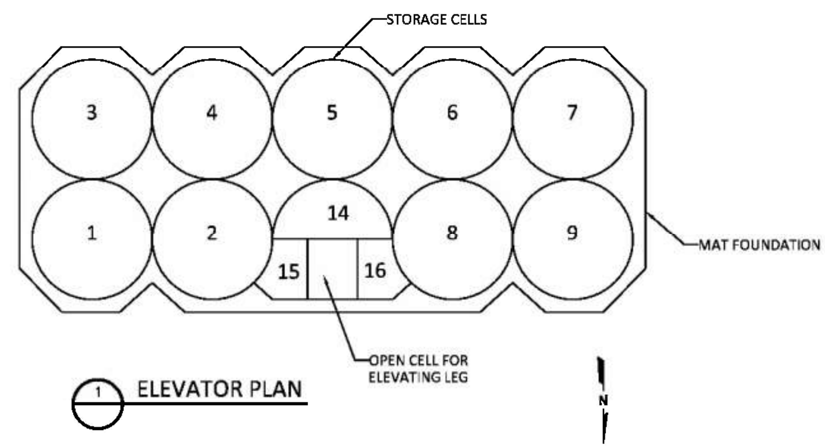
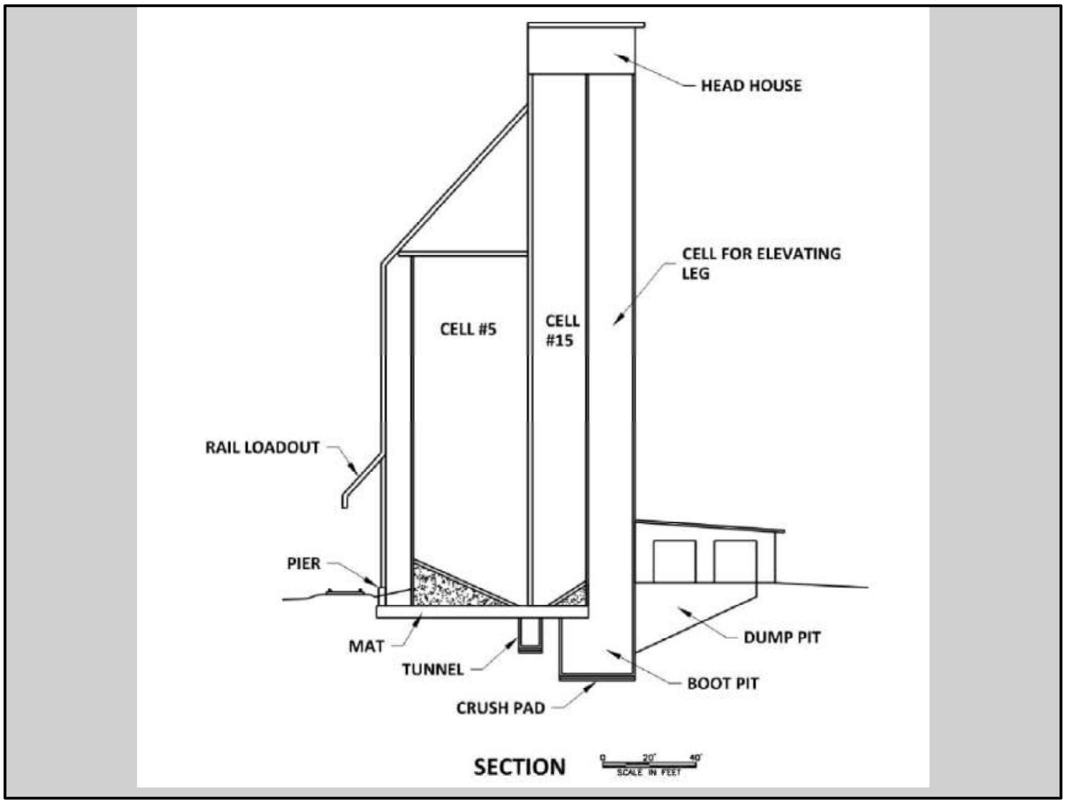
520,000-Bushel Working House

A Farmer’s Cooperative in Nebraska planned a new grain storage and handling facility along the railroad tracks. It would be a slip-formed concrete grain elevator consisting of ten cylindrical cells about 137 feet tall, arranged in two rows. The structure would be a “working house”, meaning that it would include a vertical bucket elevator called an “elevating leg”.

ELEVATOR PLAN



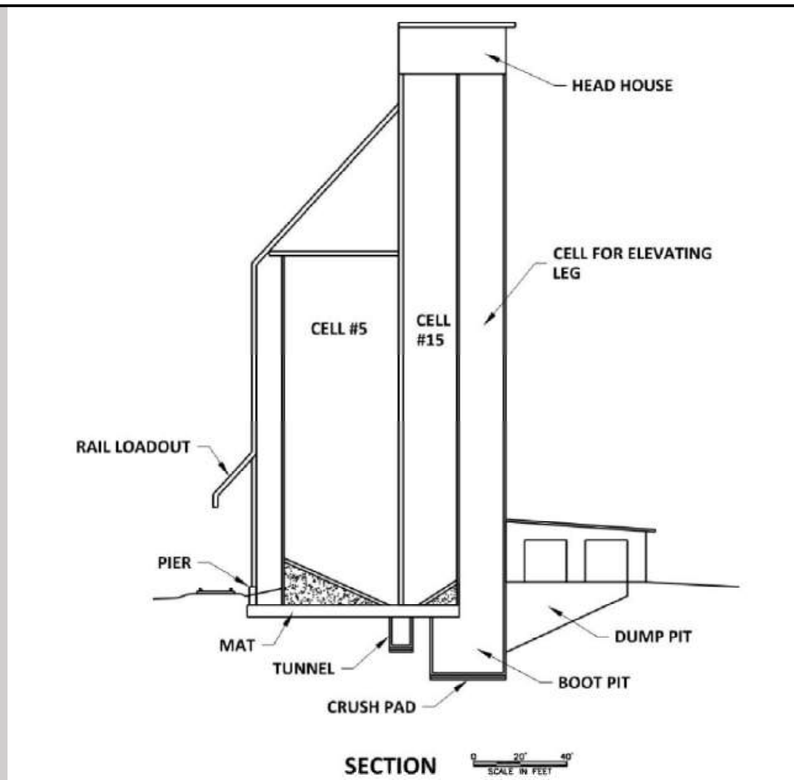
The cells would be interlocked, slip-formed concrete founded on a rigid reinforced concrete mat foundation about 2.5 feet thick bearing 4 feet below the ground surface.



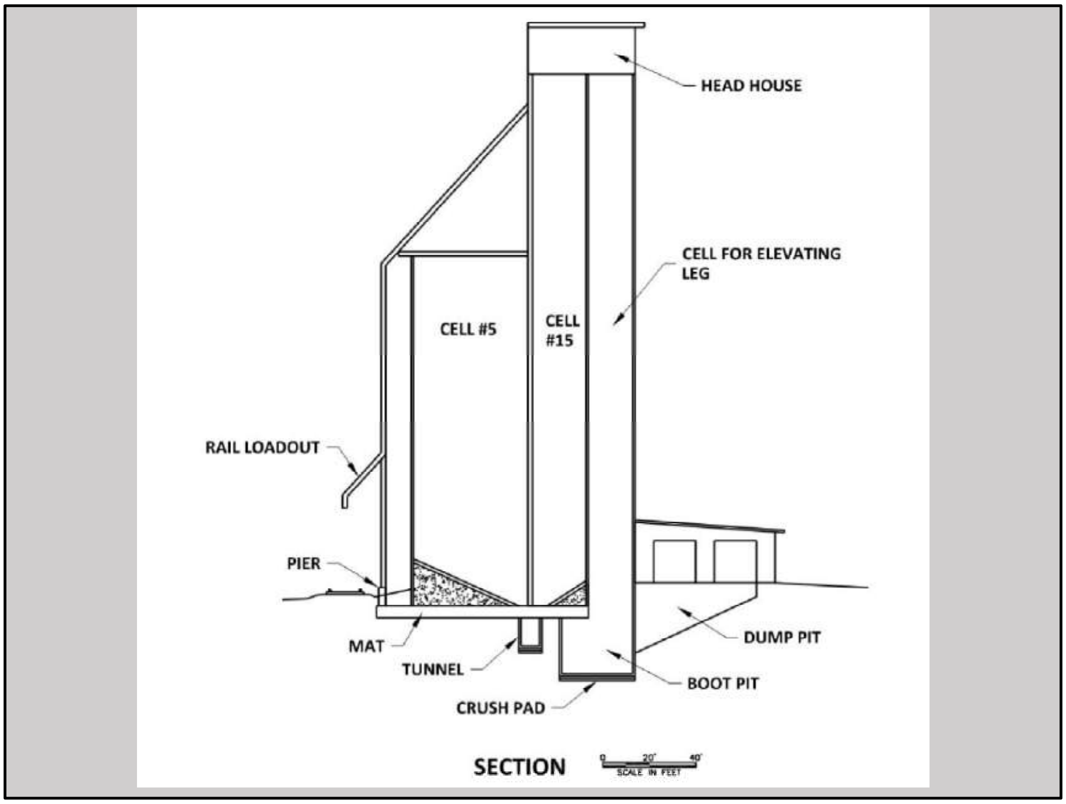
Trucks would drive through a covered dump house and dump their grain into sloping pits that would feed by gravity into the base of the elevating leg. The base of the leg would be enclosed in a "boot pit" extending 16 feet below the main foundation slab.

Grain Distributor

At the top of the leg, the grain falls into the distributor, which directs the grain into one of several chutes leading to storage cells, other storage structures, and truck and railcar loadouts. The leg can move grain rapidly to keep up with the rush of trucks bringing the harvest from the farm fields.



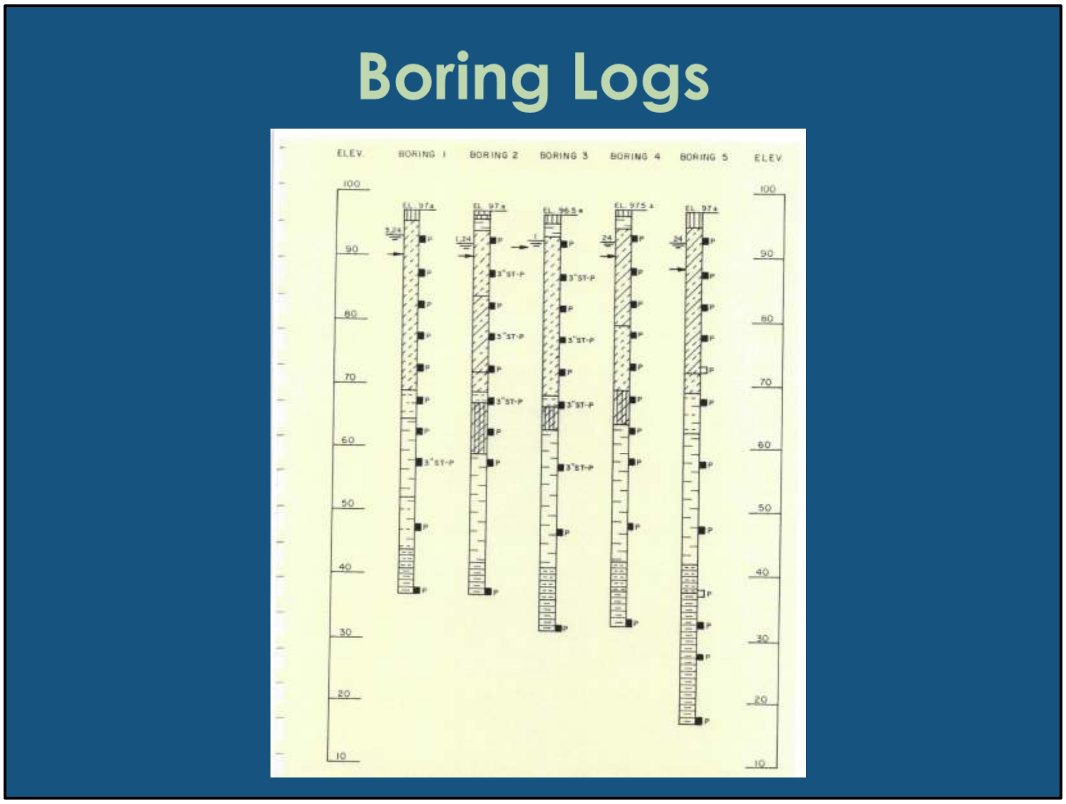
The cells would be unloaded by gravity flow onto a horizontal belt conveyor in a 7-foot-square concrete tunnel running under the center of the mat.



The tunnel and boot pit slabs were cast on bales of straw to prevent the slabs from being overloaded as the soil compressed. The mat was cast on a 12-inch-thick layer of sand to encourage vertical drainage during consolidation.

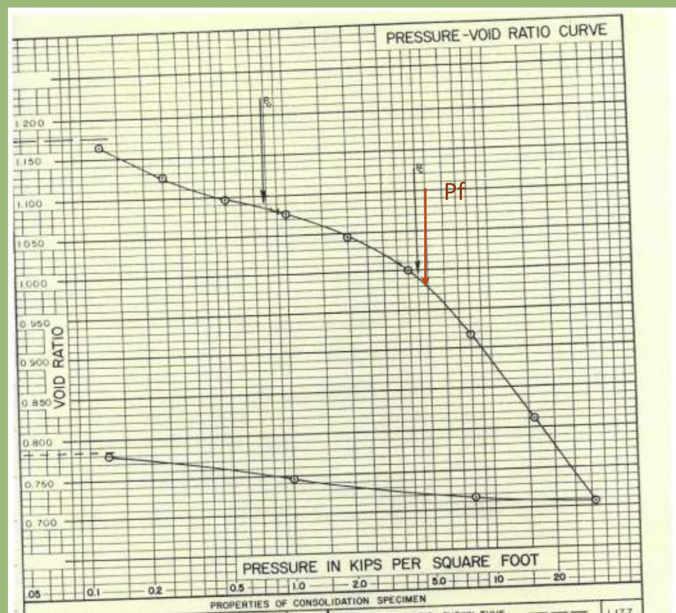
The storage capacity of the elevator would be about 520,000 bushels. The total weight of the structure and the grain would be about 52,000 kips.

The mat foundation usually extends about 2 to 3 feet beyond the edges of the cells for a gross area of about 7150 square feet. Deducting the area of the crush pads, the net foundation area would be about 6015 square feet. Filled with grain, the elevator would generate an average gross soil bearing pressure of about 8.65 kips per square foot.



The site was on a broad plain of loess underlain by glacial till. Borings showed a water table within a few feet of the ground surface, probably perched on the till. The Peorian Loess and the Loveland loess were both present, saturated, and compressible.

Consolidation Test
B-2 @ 9.8 ft.



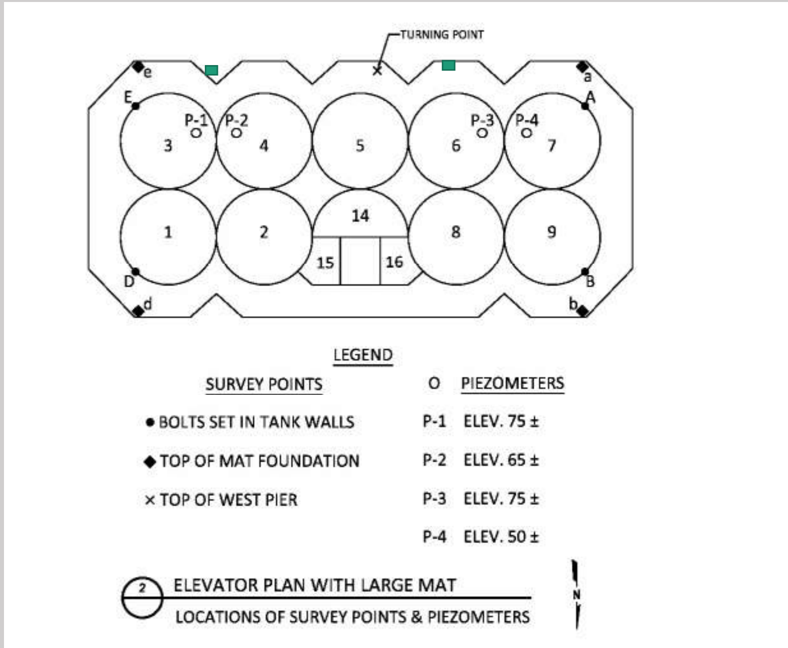
Consolidation tests showed the loess to be moderately overconsolidated, with preconsolidation pressures of about 4 to 6 kips per square foot.

When the distributed pressures from the foundation load were added to the existing overburden pressure, the vertical effective stress would slightly exceed the preconsolidation pressure. Although most of the stress change would be recompression, the settlement of the elevator would be large.

The settlement of the elevator was estimated to be 8 to 10 inches. Tilting was a concern. The recommendation was made to overexcavate the site and place about ten feet of compacted fill using the excavated silty clay (after much drying) to support the mat foundation and reduce settlement.

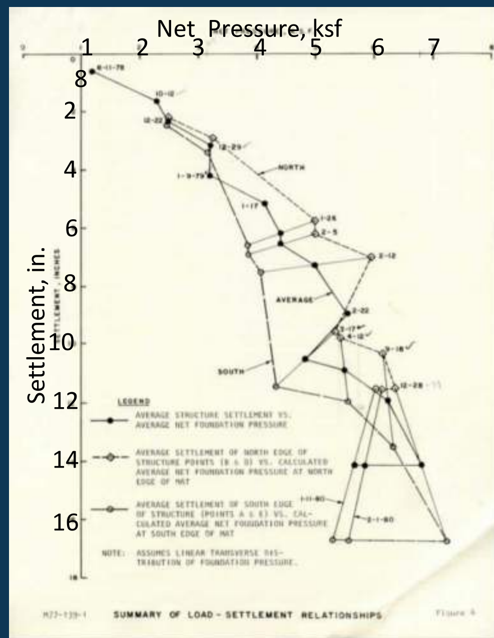
The designer suggested that the mat foundation could be enlarged by extending it 8 feet beyond the cell walls, reducing the net average bearing pressure to 6.65 ksf. This was agreed.

REVISED MAT PLAN



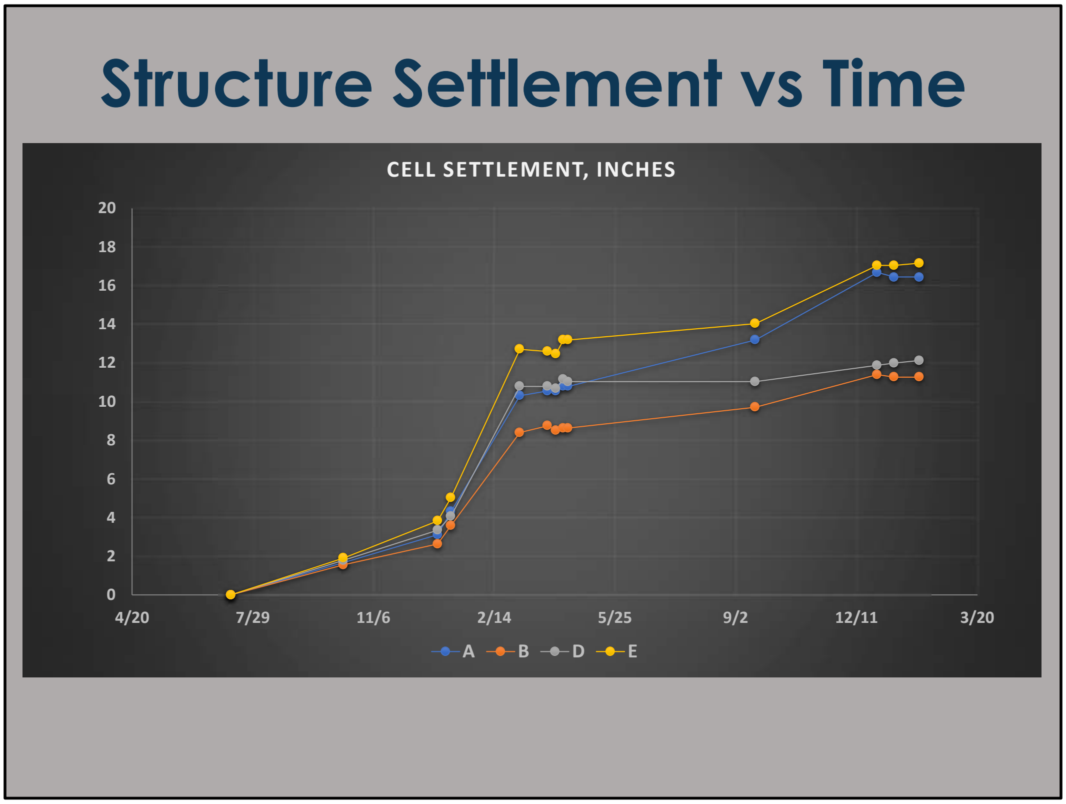
The tunnel, boot pit, and mat were constructed. Crosses were chiseled in the top of the mat near each corner, and the elevations were determined by differential levelling from a frost-free benchmark.

Settlement of Structure vs Net Bearing Pressure, ksf

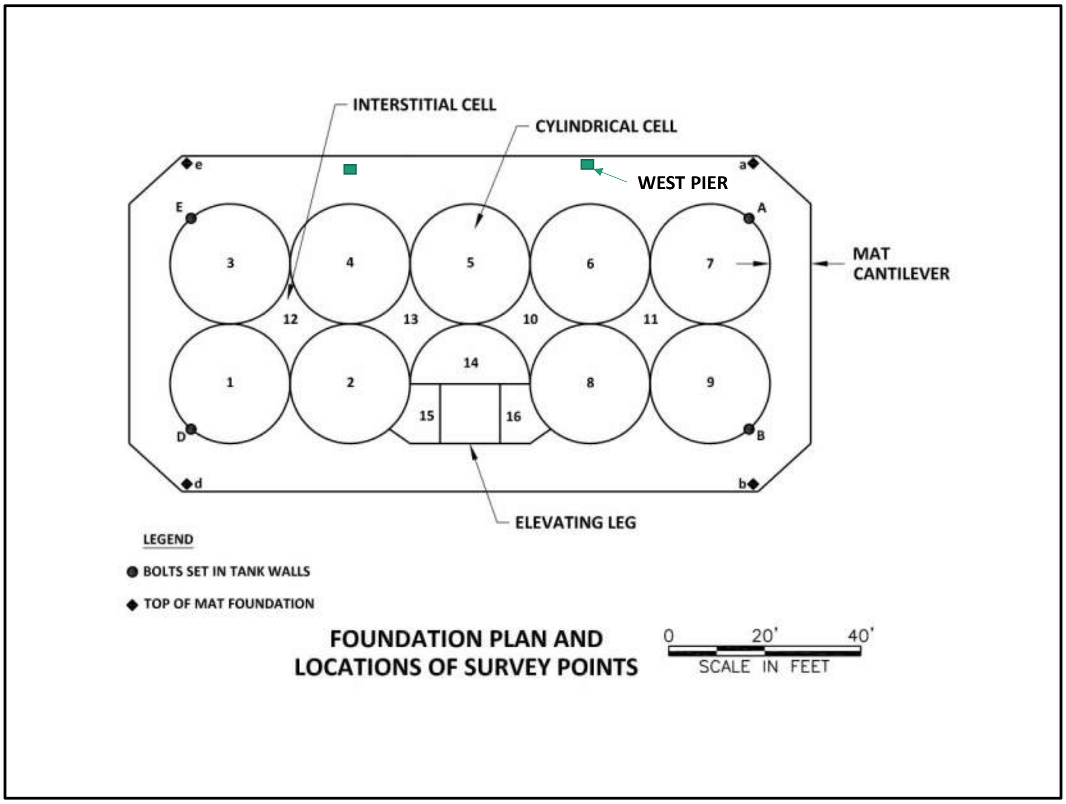


Settlement, tilting, and pore pressure changes were closely monitored. Initial filling of the structure with grain took two harvest seasons. The elevator had a persistent tendency to tilt southward. More grain was placed in the north tanks to counteract the tilt, but when all the cells were full, the average settlement was 14 inches, and the tilt was 17 inches at the top of the cells.

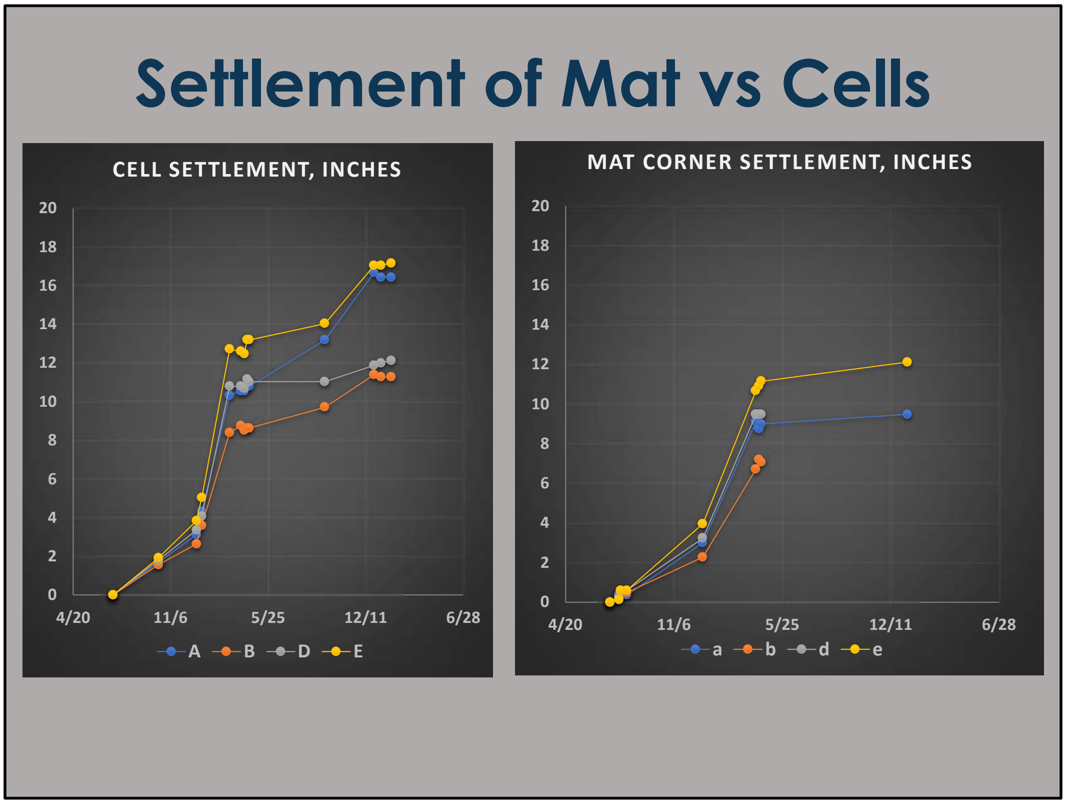
The facility was still usable.



This chart shows the settlement of the four corners versus time. The south side settled about 4 inches more than the north side.

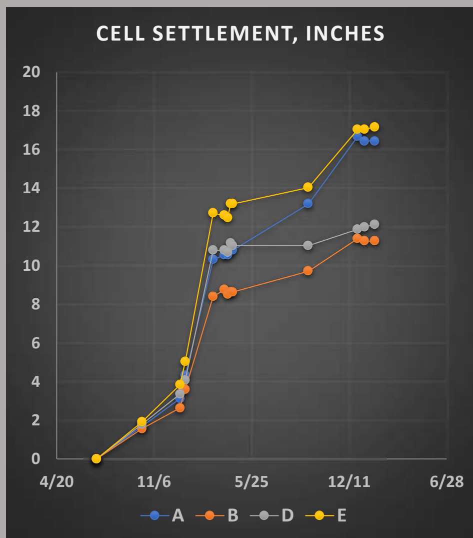


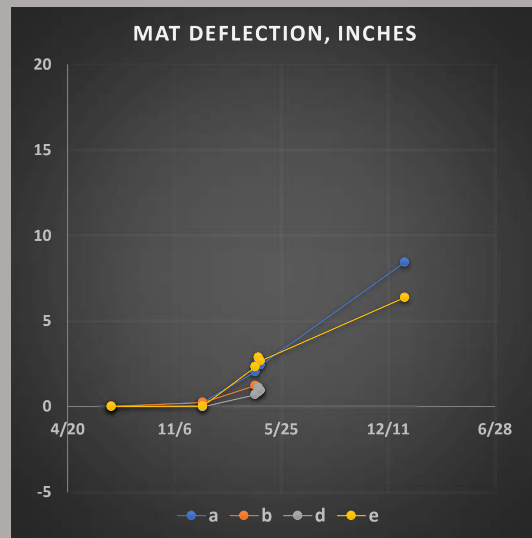
A pier supporting a railcar loading chute was used as a turning point as the surveys were looped around the structure. It was noticed that the turning point was not settling as much as the two anchor bolts on the southeast and southwest corners of the elevator, suggesting that the cantilever of the mat foundation might be deflecting upwards. Steel rods were driven to the top of the mat and surveyed as time went on.



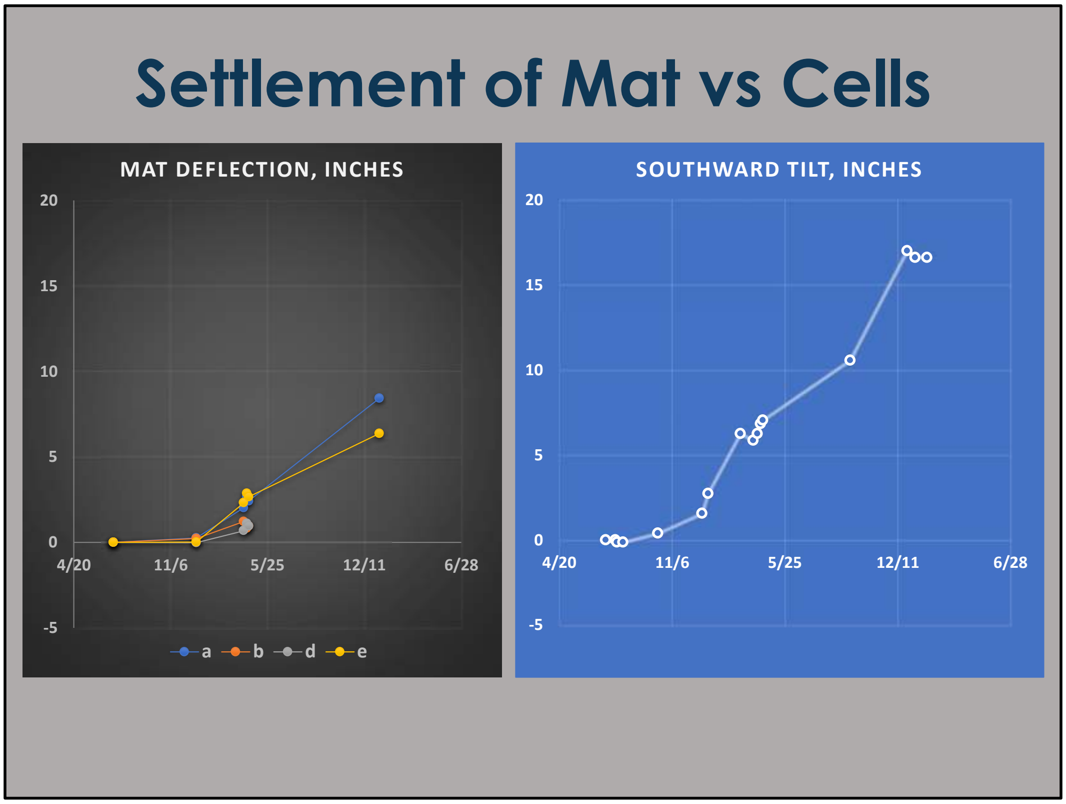
Comparison of the settlement of the mat corners to the settlement of the walls showed differences of several inches.

Settlement of Mat vs Cells





The two south corners of the mat had deflected upwards about 5.2 and 6.8 inches. This deflection must have involved a structural failure of the foundation.



The owner and designer/builder were informed that the mat was undoubtedly broken. The builder said: “We built it. We will fix it.”

Broken Mat

In the spring, when most of the grain had been removed through normal seasonal operations, the builder uncovered the foundation. The cantilever had sheared off.

Broken Mat

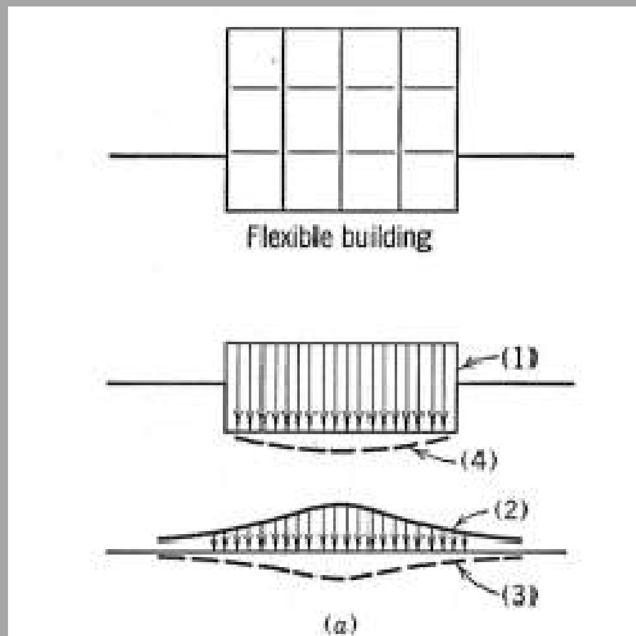
Repairs Begun

Repairs were made.

WHY DID THIS HAPPEN?

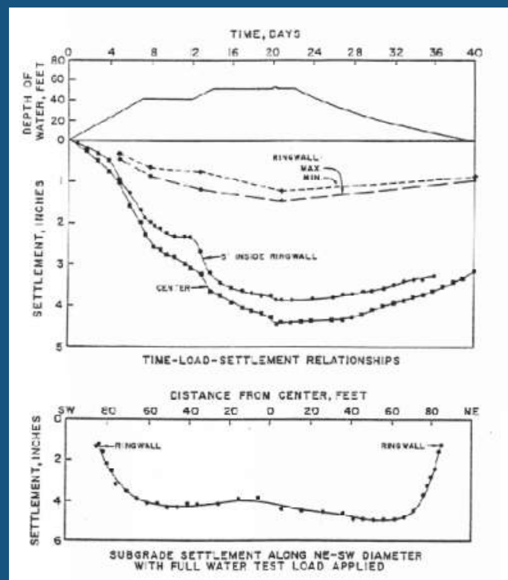
Redistribution of Foundation Contact Pressures

Stress Distribution Under Uniform Load



Elastic analysis shows us that an area loaded with a uniform pressure tends to cause higher pressure at the center and lower pressure at the edges on a plane at substantial depth. The foundation will tend to become bowl-shaped.

Settlement of Steel Ammonia Tank



An example of a uniformly loaded area is a thin-bottomed steel tank. This chart shows the measured settlement of a 170-foot diameter tank filled with 50 feet of water.

Vertical Stress Under Rigid Circular Load (Poulos, 1965)

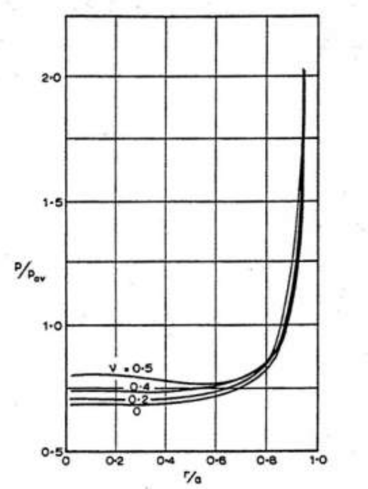


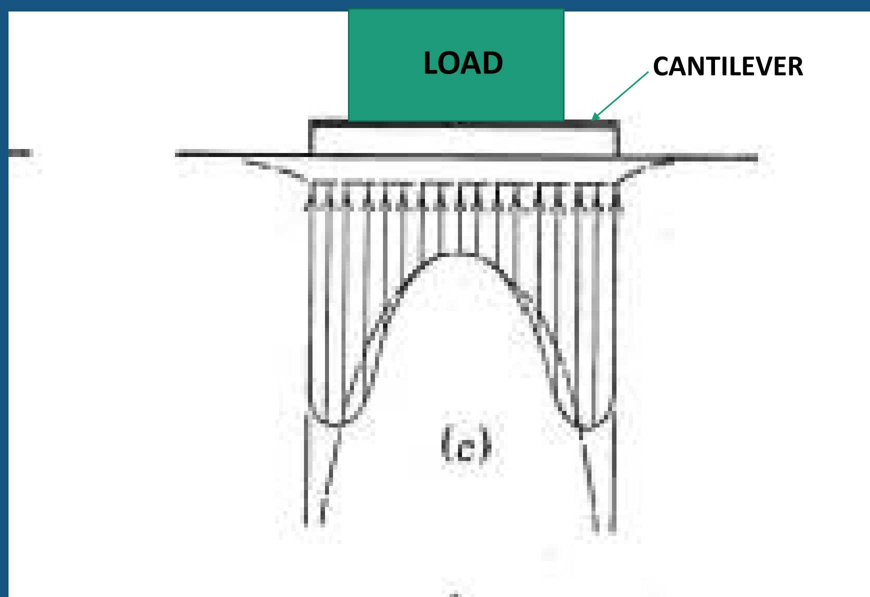
FIG.7.16 Effect of ν on contact pressure (h/a=1).

Circular Load on a Mass of Finite Thickness

From Poulos and Davis, Page 179

A rigid foundation cannot bend to conform to the dish-shaped settlement profile that a uniform load creates. As the soil compresses, the rigid body transfers load to the edges, and the applied soil pressure becomes non-uniform. Some textbooks show examples of the pressure distribution under a rigid foundation. This one is by Poulos.

Contact Pressure Under Rigid Mat



According to elastic theory, the pressure approaches infinity at the edge. This will not occur on a real soil because the soil will shear and move out from under the edge.

Still, the pressure under the outer few feet of a mat can readily exceed two or three times the average pressure, and the pressure near the center can be near one-half the average. Unfortunately, the very different stress distribution under a rigid foundation is only briefly discussed in most textbooks and design guides.

How Was The Mat Designed?

- The mat foundation was designed assuming a uniform soil bearing pressure across the mat, including the cantilevered parts outside the walls. The high pressures that developed there created bending and shear stresses that greatly exceeded the strength of the reinforced concrete.

Who Knew?

- Why is this phenomenon not widely recognized?

- The common practice in the Midwest appears to be to limit the cantilevers to about 2 to 3 feet. The thickness and reinforcing of the mat are controlled by the span across the cell, and the thickness is uniform for simplicity, so even though the soil pressure may be much higher than assumed under the cantilever, the strength of the mat is not exceeded. There may be cases where wider cantilevers have been tried and failed, but the damage is below ground and never discovered, or the failure is kept secret.

QUESTIONS AND COMMENTS?

PPHHTT IS NOT ACCEPTABLE

TALL BUILDINGS IN HOUSTON

Mats extending many feet beyond the walls have been used to support tall buildings in places like Houston where deep, compressible soils exist, and bedrock is too deep for piles. Deep basements are used to reduce settlements by offsetting part of the building weight with the weight of the excavated soil.

- Designers have used an iterative procedure consisting of a structural program with a mat on springs and a geotechnical settlement program to adjust the spring constants. The results have been heavily reinforced mats several times as thick as the one used at David City, although the gross bearing pressures are similar. Modern finite element programs may be suitable. Expert guidance is recommended.

Lessons Learned Through 54 Years of Mistakes

Collapse of Tempering Cooler

Chuck Easton, P.E.

Freese and Nichols, Inc.
Fort Worth, Texas

Lessons Learned Through 54 Years of Mistakes:

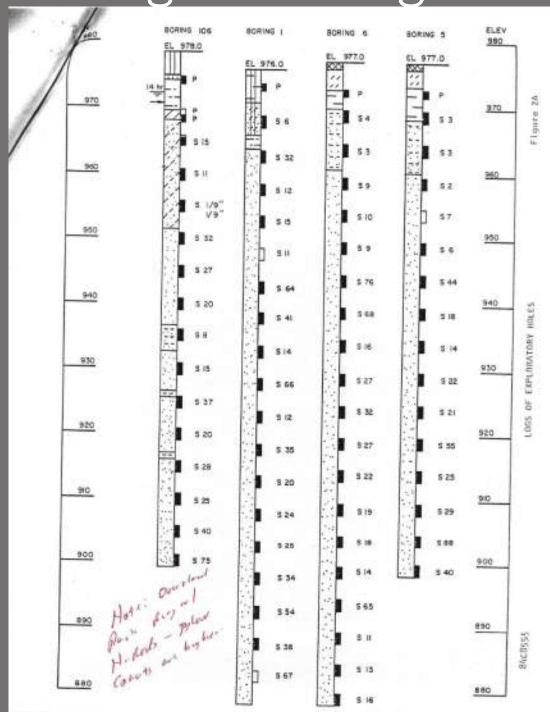
SOME EXPERIENCES WITH AUGERED PILES

Augered piles, also called Augercast piles, Continuous Flite Auger (CFA) piles, Augered Cast-in-Place (ACIP piles and several other names, are installed by drilling a hollow-stem auger to a desired depth and pumping grout or concrete down the auger to fill the hole as the auger is withdrawn. They can offer low cost, rapid construction, tensile capacity, and other benefits. They are, however, subject to several types of problems that may not be readily noticed or anticipated. The project described here encountered several problems that were dramatically demonstrated and can serve as a caution to engineers and constructors.

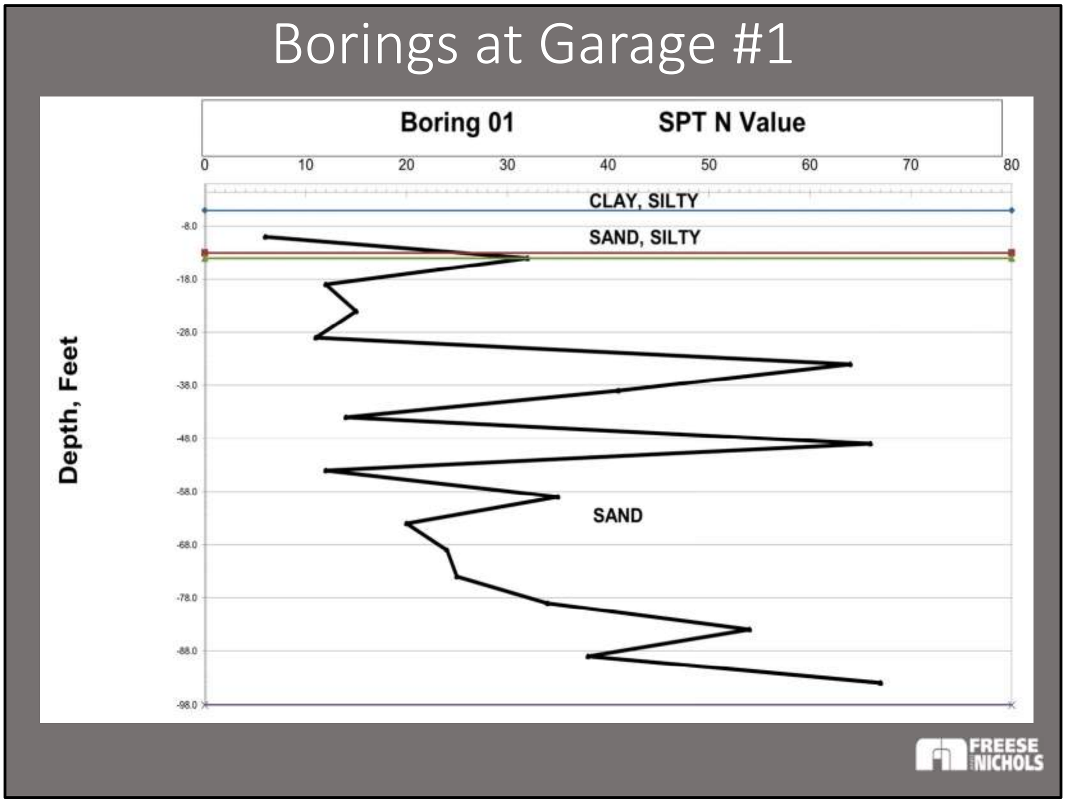
PARKING GARAGES FOR AN AIRPORT:

About 1984, Eppley International Airport wanted to build their first multi-story parking garage. It would have 6 parking levels with allowance for 2 future additional levels. Construction would use prestressed concrete columns, beams, and double-tee floor panels. Design column loads would range from about 900 to 1800 kips. Ten years later, a second garage of similar height was constructed immediately south of the first garage./

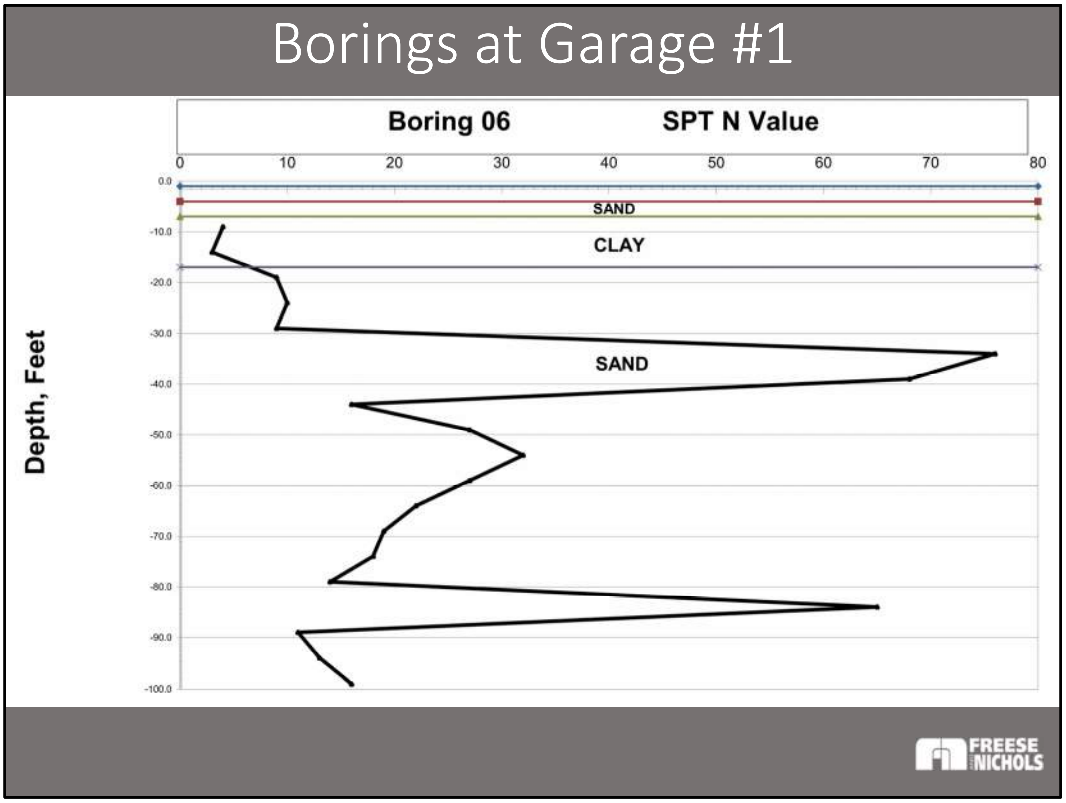
Borings at Garage #1



The airport is located in the floodplain of the Missouri River. Typical subsurface conditions consist of surface layers of weak silty and clayey soils underlain by loose to very dense fine to medium sand. At the project site, the sands extend more than 100 feet to bedrock.



As usual, SPT tests showed that the sand formations deposited by the river are highly variable in relative density. Much of the sand has SPT N values in the medium dense range, between 10 and 30 blows per foot. Other layers are dense or very dense, with a few values exceeding 70 blows per foot.



There was some indication that a dense layer might be encountered between depths of 30 and 40 feet at many column locations.

FOUNDATION RECOMMENDATIONS

LONG DRIVEN DISPLACEMENT PILES WERE RECOMMENDED:

- PRESTRESSED CONCRETE PILES
- MANDREL-DRIVEN THIN SHELLS FILLED WITH CONCRETE
- CLOSED-END STEEL PIPE PILES

LENGTHS OF ABOUT 70 TO 80 FEET WERE ANTICIPATED.

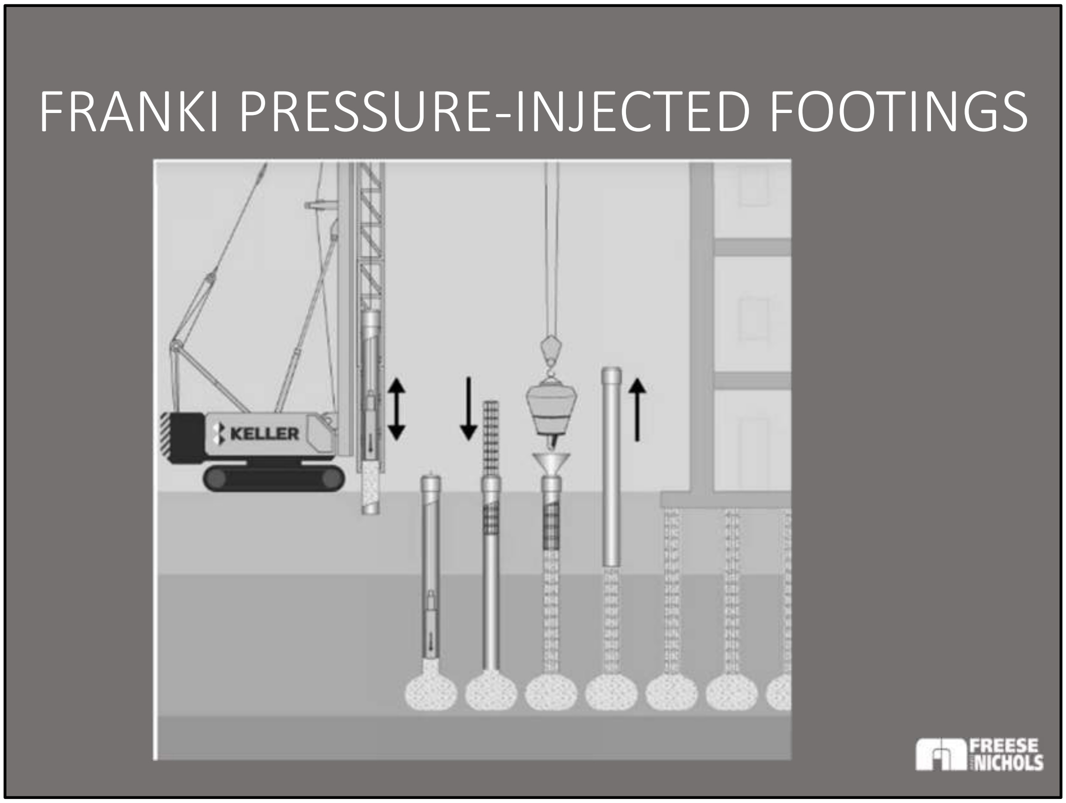
AN ALLOWABLE PILE CAPACITY OF 120 TONS WAS RECOMMENDED.

RECOMMENDATIONS WERE ALSO GIVEN FOR FRANKI PILES OR PIFS

- FRANKI PILES COULD BE MUCH SHORTER THAN NORMAL DISPLACEMENT PILES DUE TO THE WAY THEY WERE CONSTRUCTED.

- FRANKI WON THE JOB

Franki piles had been used in the eastern USA for many years. The Franki Foundation Company wanted a chance to demonstrate their advantages in the Midwest. They were willing to absorb the mobilization cost.



Franki PIFs are installed by driving a heavy steel casing using a 5-to- 10-ton cylindrical drop hammer.
The casing is open at the bottom. A few feet of zero-slump concrete is placed in the bottom and driven with the hammer. Friction engages the casing and pulls it down. At the desired depth, the casing is restrained. Concrete is added a few cubic feet at a time and driven partially out of the casing, forming a bulb that compacts and stresses the surrounding sandy soil. The casing is then filled with concrete and withdrawn, forming a shaft.

FRANKI PIFS WERE INSTALLED TO SUPPORT GARAGE #1
LOAD TESTS CONFIRMED THE CAPACITY
OUR OFFICE MONITORED THE FOUNDATION PERFORMANCE

PARKING GARAGE NO. 1
FRANKI PRESSURE INJECTED FOOTINGS
PILE GROUP SETTLEMENT UNDER LOAD

COLUMN	DEAD LOAD	NO. OF PILES	LOAD PER PILE, TONS	SETTLEMENT, INCHES
6A	515	3	86	0.54
6B	800	6	66	0.53
6C	800	6	66	0.42
13A	515	3	86	0.42
14A	585	3	98	0.47
14B	890	5	89	0.38
14C	890	5	89	0.32
14D	890	5	89	0.29

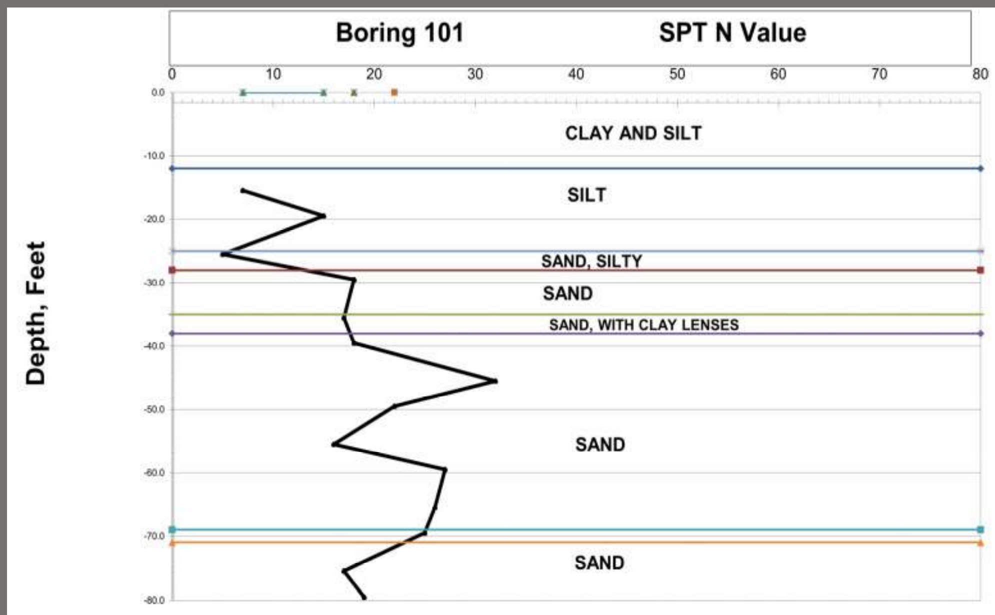
FREESE NICHOLS

The geotechnical engineer and staff member surveyed the pile caps before and after the garage was erected. The performance of the foundation was thus documented.

PARKING GARAGE #2

Ten years after construction of Garage #1, a second garage of similar height was constructed immediately south of the first garage.

BORINGS FOR PARKING GARAGE #2



Six additional borings were drilled.

FOUNDATION SELECTION

THE FRANKI COMPANY HAD LEFT THE VICINITY, SO PIFS WERE NOT CONSIDERED

LONG DRIVEN DISPLACEMENT PILES WITH AN ALLOWABLE CAPACITY OF 120 TONS WERE RECOMMENDED:

- PRESTRESSED CONCRETE PILES
- MANDREL-DRIVEN THIN SHELLS FILLED WITH CONCRETE
- CLOSED-END STEEL PIPE PILES

THE SUCCESSFUL BIDDER SELECTED PRESTRESSED CONCRETE PILES, BUT THEN PROPOSED TO USE AUGERED PILES FOR A CREDIT.

OUR OFFICE REVIEWED THE PROPOSAL AND CONCLUDED THAT 18" DIAMETER AUGERED PILES COULD DEVELOP THE REQUIRED CAPACITY.

THE OWNER ACCEPTED THE PROPOSED CHANGE.

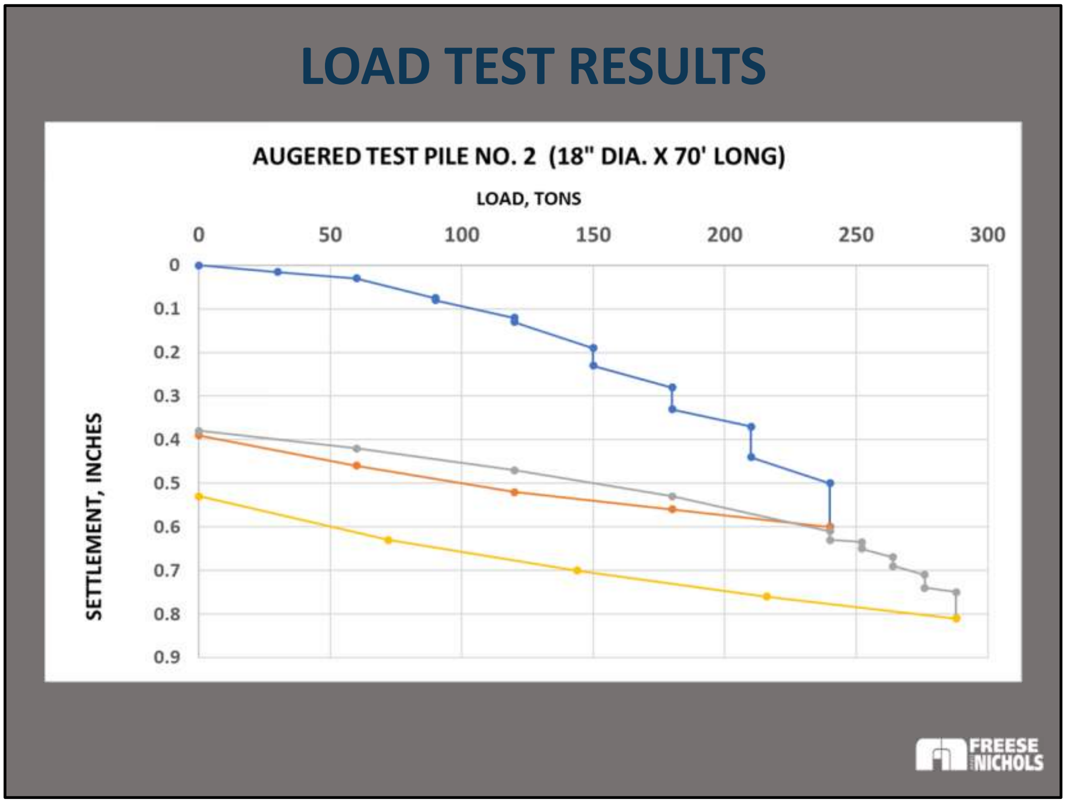
FREESE NICHOLS

DRILLING

Augered piles are installed by drilling with a hollow-stem auger. The opening at the bottom of the auger is plugged with a cork. When the desire depth is reached, the grout pump is started, the auger is raised a few inches to make room for the cork to be expelled, and the grout is pumped to fill the hole as the auger is slowly withdrawn.

Grouting

The foreman watches carefully to be sure the auger is not raised too fast, which could allow soil to enter the pile.



Test piles confirmed that the desired 120-Ton capacity could be obtained.

FIRST PROBLEM

INSPECTION REPORT #2

"…The auger plugged and was withdrawn from the hole. The contractor removed the trap door from the bottom of the augers and began using corks."

INSPECTION REPORT #5

"when grouting was attempted, there was a blockage in the line, and the auger had to be completely withdrawn. After grout flow was achieved the auger was reinserted and drilled to a depth of 80 feet, approximately 8 feet deeper than before."

FREESE NICHOLS

Problems with installation of production piles began right away.

This happens occasionally on projects, and it was our practice to require that the pile be lengthened by 10 feet to compensate.

FIRST PROBLEM

THE AUGER OR GROUT HOSE PLUGGED, AND THE AUGER WAS WITHDRAWN WITHOUT GROUTING FIFTEEN TIMES DURING THE COURSE OF THE PROJECT.

IN SATURATED CLEAN SAND,THE SAND CAVES INTO THE HOLE WHEN THE AUGERS ARE WITHDRAWN, CAUSING SOME DISTURBANCE TO THE SUROUNDING SAND.

FREESE NICHOLS

INVESTIGATION

A BORING WAS DRILLED ABOUT 5 FEET FROM ONE OF THE PILES THAT HAD BEEN RE-DRILLED AND RE-GROUTED BECAUSE THE AUGER WAS PLUGGED.

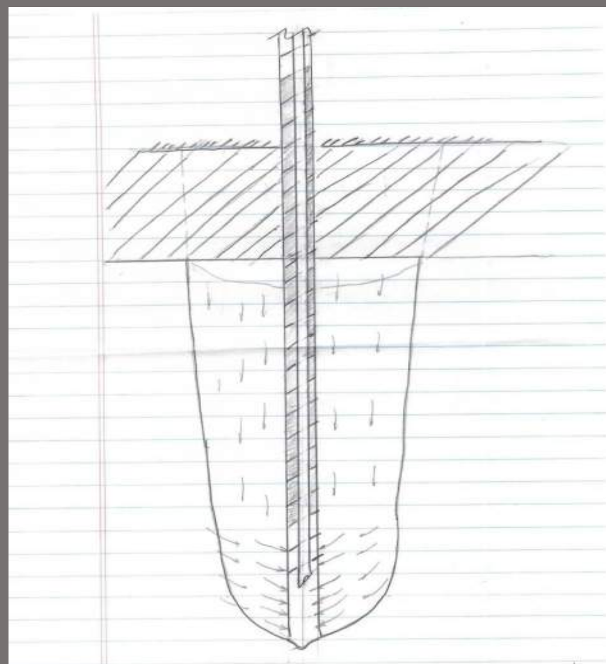
SPT N VALUES IN THE SAND ALL WERE 10 OR LESS, CLASSIFIED AS LOOSE.

FREESE NICHOLS

Because the clogged auger condition occurred so many times, we wanted to determine how much the sand was altered at previously-placed adjacent piles.

It appears that the auger removal completely loosened the sand in a column at least 10 feet in diameter.

Apparent effect of Pulling Auger without Grouting



We concluded that withdrawing the auger without grouting loosened the sand to a distance of more than 5 feet away.

ADDITIONAL INVESTIGATION

A PILE WAS INSTALLED 4.5 FEET FROM THE CENTER OF TEST PILE #2.

THE AUGER WAS ADVANCED TO 70 FEET AND WITHDRAWN, ADVANCED TO 80 FEET AND WITHDRAWN AGAIN, THEN RE-ADVANCED TO 80 FEET AND GROUT WAS PLACED.

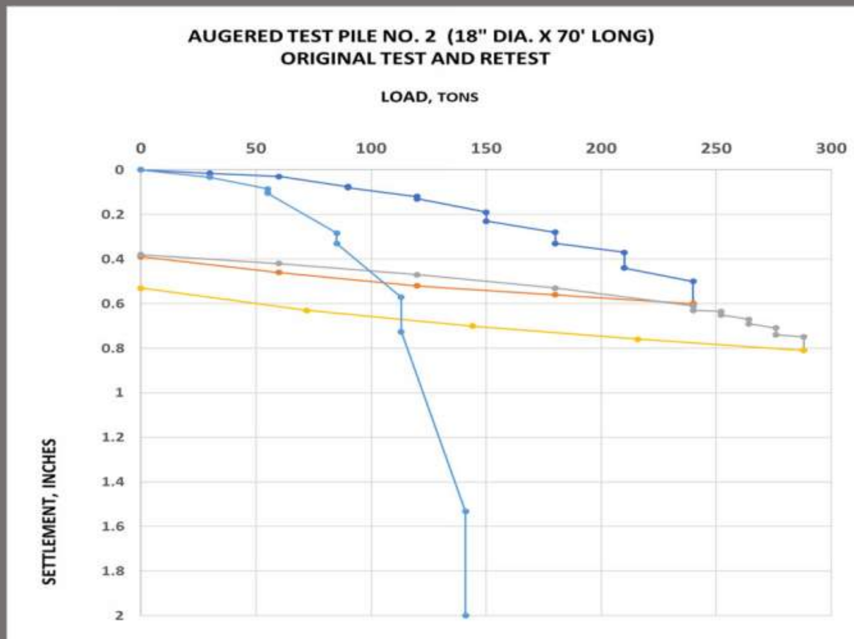
We were concerned about the effect of auger withdrawal on previously-placed piles in the same group. This sequence had occurred several times when other piles in the group were already in place.

ADDITIONAL INVESTIGATION

AFTER THE NEW PILE HAD CURED, TEST PILE #2 WAS AGAIN LOAD TESTED.

THE RESULTS ARE SHOWN ON THE NEXT SLIDE.

LOAD TEST RESULTS



Guess which trace is the retest.

NEW STRUCTURE MEETS OLD ONE

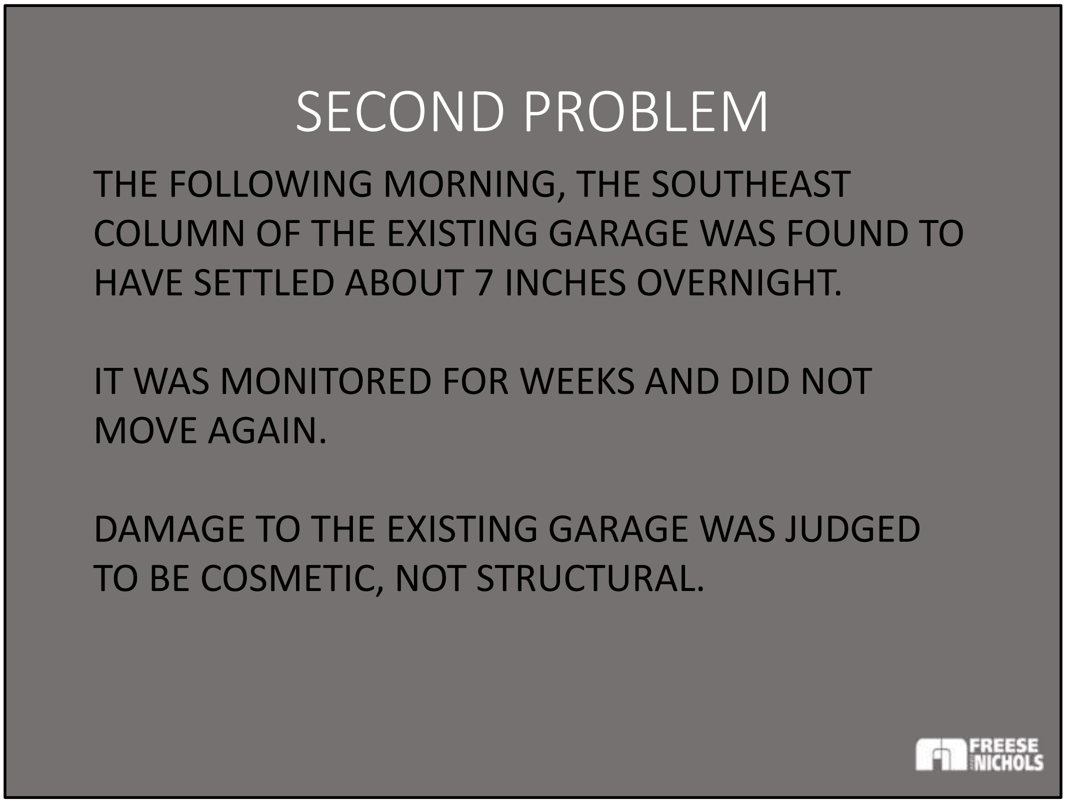
Because the two garages were combined, new column foundations were needed adjacent to the existing structure.
The column spacings were different, so most of the new columns were offset from the existing ones. At the southeast corner of Garage #1, the two foundations would be adjacent.

SECOND PROBLEM

ON JUNE 19, THE FIRST PILE IN THE GROUP ADJACENT TO THE SOUTHEAST CORNER OF THE EXISTING GARAGE WAS PLACED WITHOUT INCIDENT.

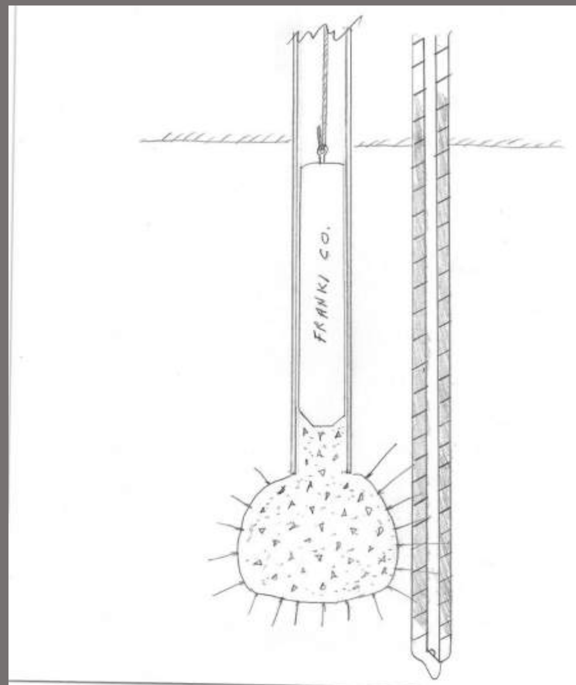
ON JUNE 22, THE SECOND PILE OF THAT GROUP WAS PLACED, AND THE GROUT HEAD WAS SHORT. IN ACCORDANCE WITH THE ENGINEER'S RECOMMENDATION, THIS PILE WAS RE-AUGERED AND RE-GROUTED.

The criteria required that grout appear at the ground surface while the auger tip was at least 15 feet deep. The remedy was standard practice.



The settlement was clearly due to the installation of the new augered piles close to the existing Franki PIFs.

Augered Pile Installed Adjacent to Franki Pile



The settlement was clearly due to the installation of the new augered piles close to the existing Franki PIFs.

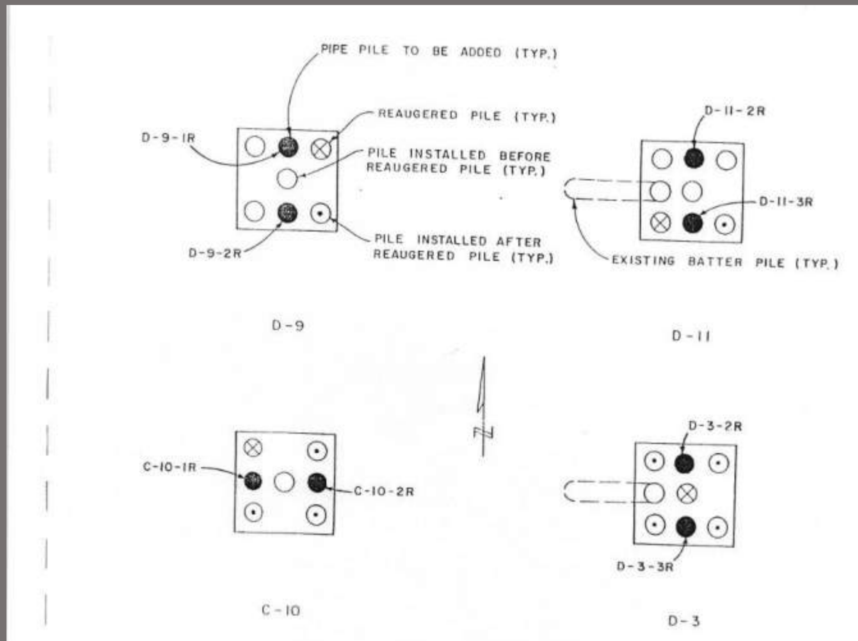
THE MISTAKES

THE POTENTIAL FOR DISTURBING THE EXISTING FRANKI PILES WAS NOT RECOGNIZED.

THE EFFECT ON PREVIOUSLY INSTALLED AUGERED PILES WHEN THE AUGER WAS REMOVED WITHOUT GROUTING WAS UNDERESTIMATED

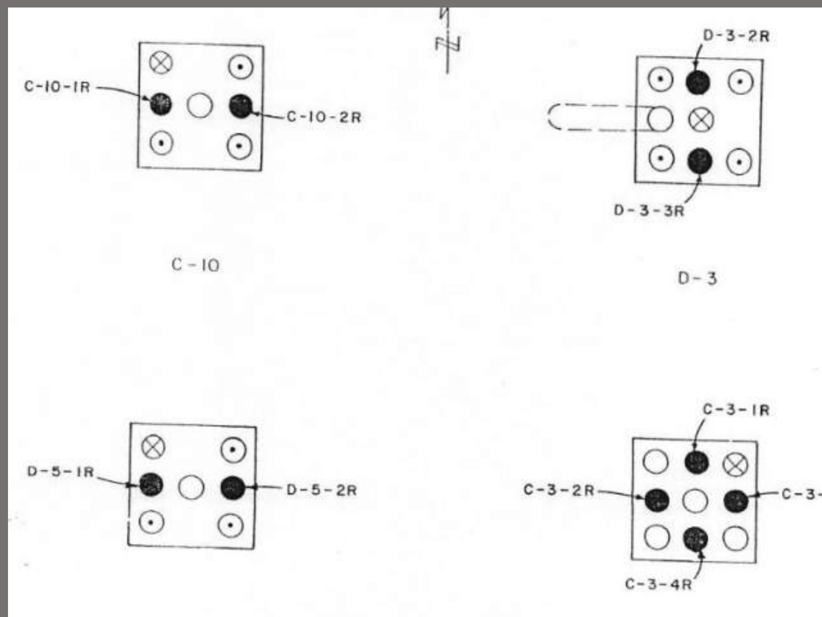
FREESE NICHOLS

ASSUMED EFFECT OF PULLING AUGER



Previously placed piles adjacent to a re-augered pile were assumed to have negligible capacity. Piles added after the re-augered pile were assumed to have full design capacity. Steel piles were added to make up the lost capacity and provide balanced support. It was assumed that the heavy steel piles bearing on rock could provide more than 120 tons support if needed.

ASSUMED EFFECT OF PULLING AUGER



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EVALUATION AND REMEDIES

MANY PILES IN MANY GROUPS WERE CONSIDERED TO HAVE SHARPLY REDUCED CAPACITIES DUE TO AUGER REMOVAL.

ADJACENT PILES WERE ASSIGNED CAPACITIES LESS THAN ONE HALF THE DESIGN CAPACITIES.

STEEL PIPE PILES WERE LOCATED IN PLACE OF OR BETWEEN THE AUGERED PILES AND DRIVEN TO BEDROCK AT DEPTHS RANGING FROM ABOUT 116 TO 147 FEET.

FREESE NICHOLS

Distribution of the added cost was negotiated in a settlement.

Some Lessons Learned Through 54 Years of Mistakes:

MISSOURI RIVER BRIDGE COFFERDAM

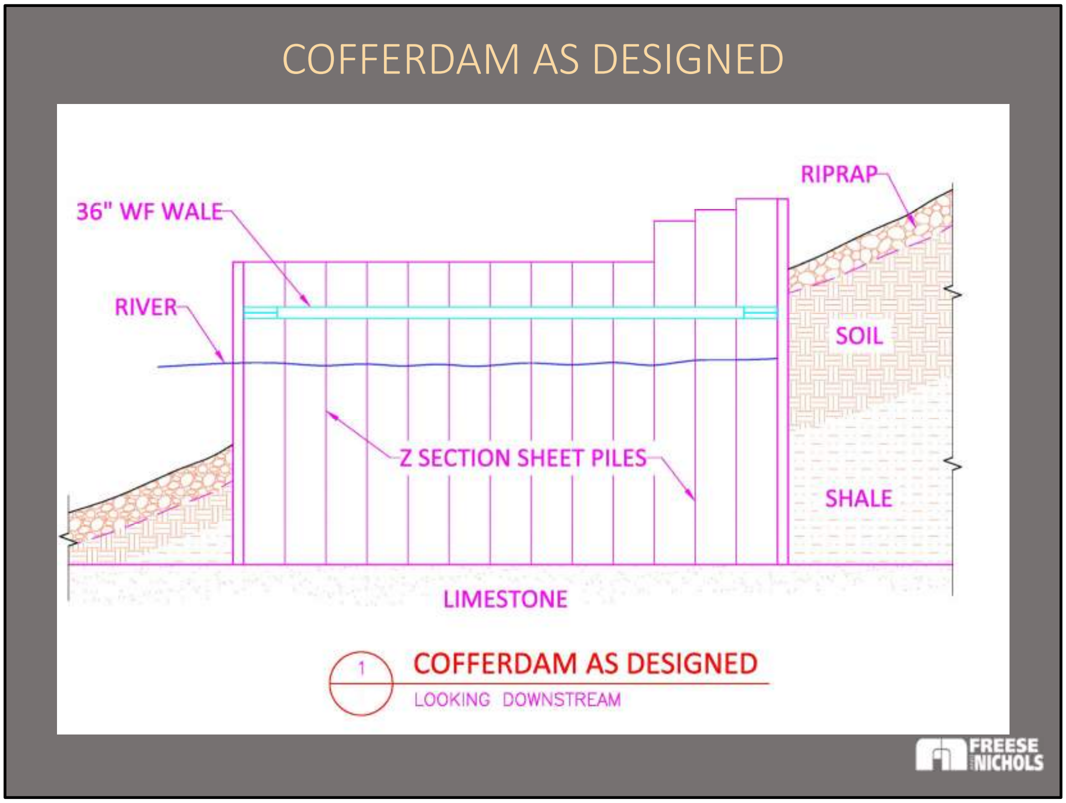
Occasionally the best-laid plans for construction get sidetracked by what might be thought to be a minor oversight.
This was the case during construction of a new bridge across the Missouri River for State Highway #2 at Nebraska City, Nebraska.

The topography and geology varied greatly along the length of the proposed bridge. The Nebraska (west) side was a high bluff with limestone bedrock just a few feet below the ground surface. Shale and limestone were found about 16 feet below the water surface at the west bank. The Iowa side was a floodplain with a few feet of clay and silt at the surface, underlain by up to100 feet of sand over bedrock.

NEBRASKA CITY BRIDGE
Oblique View Looking Downstream

A shallow footing on limestone was selected for the west abutment. The mid-river and east bank piers were built on cased drilled shafts through alluvium and into bedrock. Several piers were needed on the Iowa floodplain, and steel H piles were driven to bedrock there.

The west bank pier was to be a footing on limestone beneath several feet of shale about 22 feet below summertime water level. A temporary cofferdam would be built and dewatered for construction of the footing.



The cofferdam would be about 40 by 60 feet in plan. It was to be built using Z-section steel sheet piles driven through the shale to the limestone. It would be braced using 36-inch wide-flange beams serving as wales and struts.

A NEW DEVELOPMENT

My boss had visited the project. He called me into his office.

The Contractor had tried to dewater the cofferdam, but water was entering too rapidly.

A worker had checked the bottom using SCUBA gear and reported that water was entering through cracks in the limestone.

Could I dive in the cofferdam and determine what was wrong?

I had taken a diving class and done considerable shallow-water diving in college –16 years before.
The worker (a pile buck) had been an assistant diving instructor, and he brought me up to speed on the new equipment.

ADVENTURE!

The worker helped me rent SCUBA gear and showed me how to use features new to me.

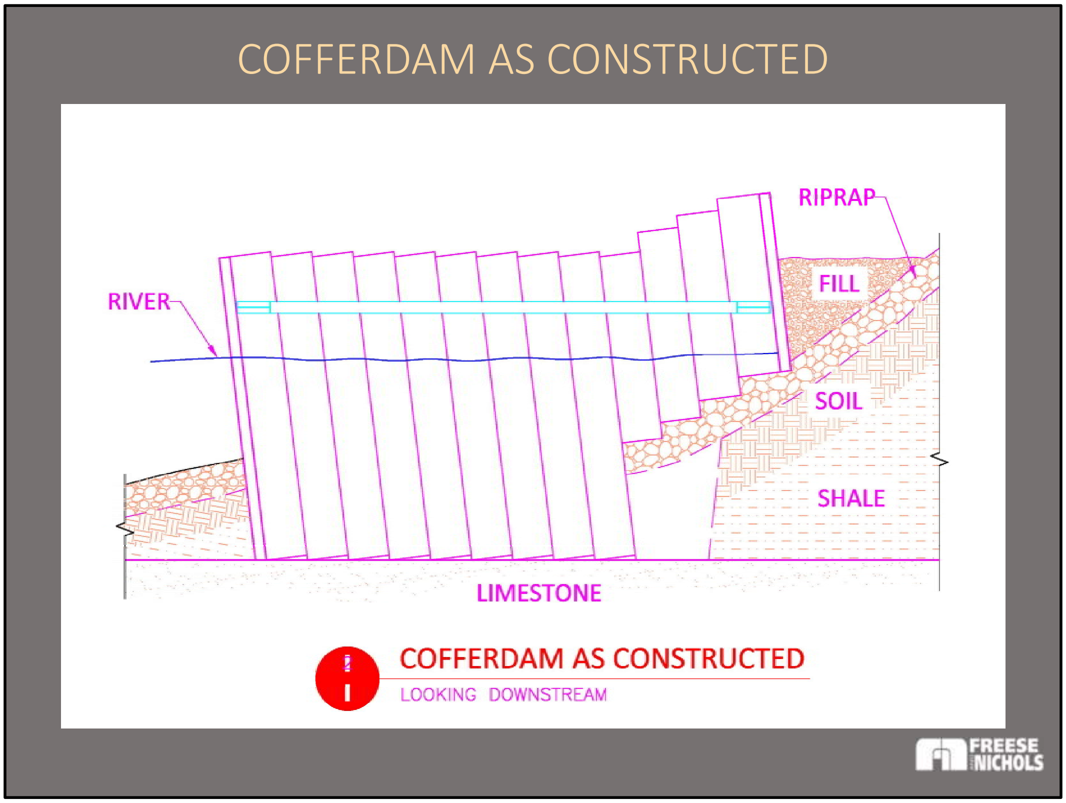
We dived in the cofferdam and found that the water was completely black!

By feeling along the edges of the excavation, we determined what was wrong.

We could see nothing. Flashlights did not help. It was bit of a psychologic challenge for me.

COFFERDAM AS CONSTRUCTED

Note that the sheet pile tops are higher near the left end of the downstream side.



The cofferdam had wracked and tilted towards the river. The excavation was short of its design width on the landward side. Some sheet piles had refused on the riprap.

THE PROBLEM

Sheet piles had refused on the riprap. They never reached the shale.

The contact with the riprap was quite leaky.

Because the cofferdam leaned towards the river, the backhoe had not reached the intended back edge of the footing location.

More excavation needed to be done along the shoreward side, the riprap needed to be removed, and the sheet piles needed to be driven down.

FREESE NICHOLS

THE MISTAKE

The sheet piles were of random lengths. No one had measured and recorded their lengths.

Apparently, no one realized that some sheet piles had not reached the limestone.

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It is common practice to mark the pile length near the top with a paint stick

THE LESSON

Always measure and mark the piles before driving.

Keep the objective in mind and stay alert.

FREESE NICHOLS

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THE SOLUTION

Hard-hat divers worked in the excavation for 2 or 3 months

They helped widen the excavation and cleaned the bottom to serve as a bearing surface,

I was asked to make the final inspection to confirm the size and condition of the bearing surface.

It was December. The temperature was below freezing.

I rented a dry suit and took a lesson in its use.

FREESE NICHOLS

FINAL INSPECTION

The water was cold, but I wore warm clothes under the dry suit.

YUP, IT WAS COLD

FREESE NICHOLS

I swept the bottom with my hand while following a pipe across the excavation. The pipe was moved, and the procedure was repeated until the entire surface had been examined. Footing corners were marked using a plumb pipe, and I confirmed that the entire bearing surface was ready

THE SOLUTION

Several feet of concrete was placed in the bottom using a tremie.

The cofferdam was dewatered, and the footing and pier were built.

TREMIE AND HOPPER

FREESE NICHOLS

CONCRETE PUMP

CONCRETE PUMP

Some Lessons Learned Through 54 Years of Mistakes:

MISSOURI RIVER BRIDGE COFFERDAM

Questions, Comments?

CHUCK EASTON. P.E.
cne@freese.com
817-729-5969