CONVERSION OF POST-TENSIONED SLAB-ON-GROUND TO PILE-SUPPORTED STRUCTURAL FOUNDATION

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One type of post-tensioned slab-on-ground foundation repair is to lift the foundation off the soil and make it a structurally suspended foundation. This paper describes the construction methods, analysis, code requirements, and load testing.

KEYWORDS

deflection; expansive clays; hydraulic ram; load test; pile-supported foundation; post-tensioned slab-on-ground; ribbed foundations; stiffening ribs; underpinning piles.

INTRODUCTION

In Texas and other parts of the southwest United States, post-tensioned slab-on-ground foundations are the most common type of residential foundations. In most of these areas, the soils are varying degrees of expansive clays. In Texas, post-tensioned foundations are ribbed foundations that consist of a uniform-thickness slab with stiffening ribs in both directions. In some cases (less than 1%), these foundations experience unacceptable movement and foundation repairs are necessary. In severe cases, the most effective repair is to lift the foundation out of the soil and convert it to a suspended pile-supported foundation.

To convert the foundation, underpinning piles are placed under the perimeter and interior of the foundation. Then, the foundation is lifted out of the soil so that it is no longer influenced by or supported by the soil. The foundation that was originally uniformly supported by the soil is now supported by the underpinning piles and functions as a suspended foundation. This type of repair has been successfully used in Texas for nearly 20 years.

The decision to elevate the foundation is dependent on factors such as excessive uniform tilt (approximately 1% slope), excessive deflection (approximately $L/360$) or when the future soil movement is unknown or expected to be excessive (Fig. 1 and 2). The homeowners are able

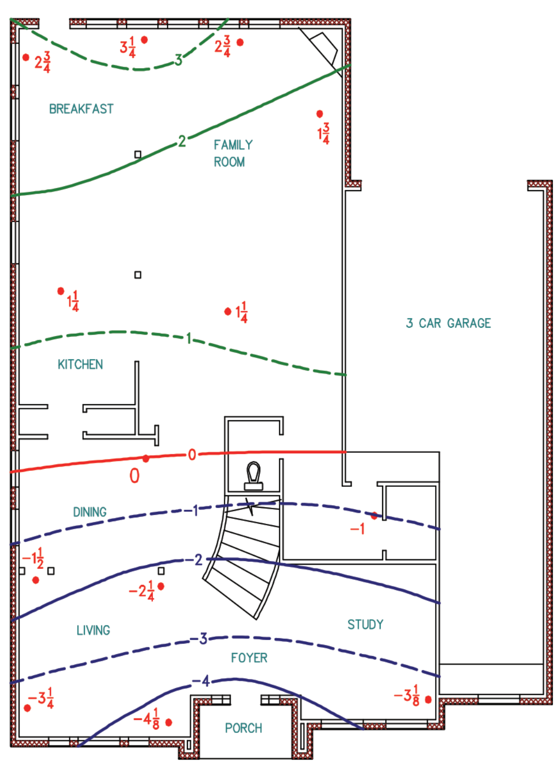

Fig. 1—Excessive floor slope.

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to live in the residence while the foundation is being repaired. The plumbing is monitored and tested after the lifting process.

MATERIALS

The as-designed foundations are usually 4 in. (100 mm) thick slabs, ribs 8 to 12 in. (200 to 300 mm) wide and beams 24 to 30 in. (610 to 760 mm) deep, with spacing at 10 to 12 ft (3.0 to 3.7 m) in each direction (Fig. 3 and 4). Post-tensioning strands are spaced 4 to 5 ft (1.2 to 1.5 m) in each direction in the slab and there are one or two strands draped into each beam. Concrete is usually specified as $f_c' = 2500$ to 3000 psi (17 to 21 MPa). Experience has shown these dimensions are very conservative because the as-constructed foundation cross section and concrete strength are usually greater than specified (Fig. 5).

The pilings are 12 in. (300 mm) long interlocking segments of 2.375 in. (60 mm) OD steel pipe (Fig. 6). To hold the pipe together and prevent uplift of the upper sections, a steel cable is inserted in the center of the pipe, then stressed and grouted. This provides a minimum tensile capacity of 10 kip (44 kN). The piles are installed one pipe at a time with a hydraulic ram pushing against the weight of the foundation (Fig. 7). Piles are driven to refusal, which is either friction resistance in clay or end bearing on a hard layer of rock. Terminal drive force is approximately 45,000 lb (200 kN), which is at least twice the final lifting force. The normal depth is 20 to 35 ft (6.1 to 10.7 m) with a minimum depth of 15 ft (4.6 m). This installation procedure satisfies the load test requirements of IBC Sec.1810.3.3. After all the piles have been installed, the foundation is gradually

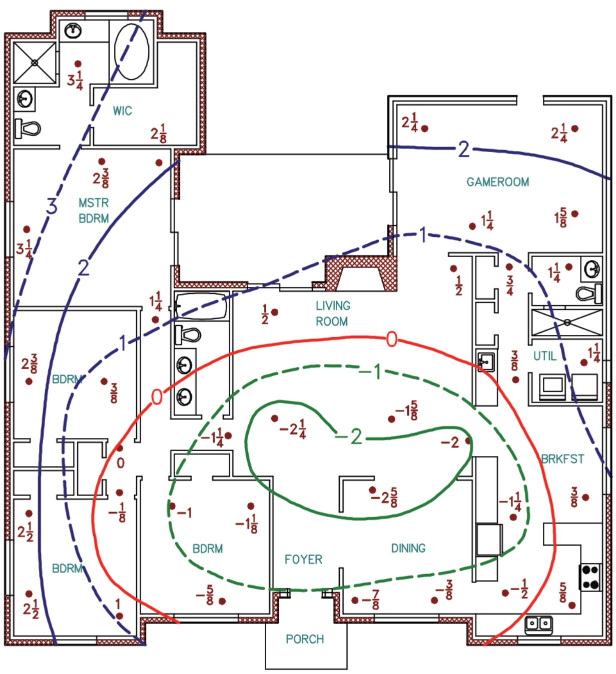


Fig. 2—Excessive deflection.

Fig. 3—Typical ribbed foundation ready for concrete placement.

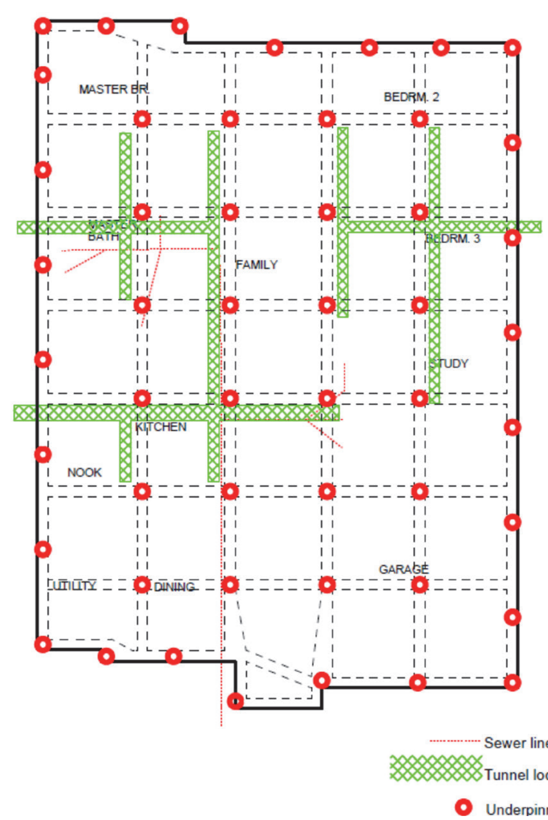


Fig. 4—Foundation plan showing grade beams, underpinning piles, tunnel locations, and sewer lines.

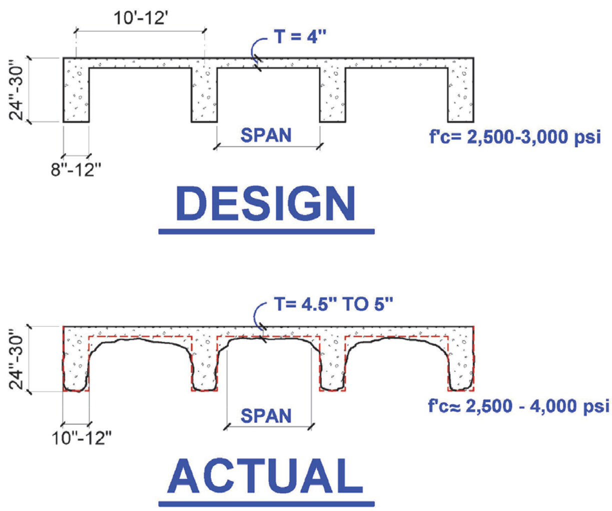


Fig. 5—As-designed versus actual construction.

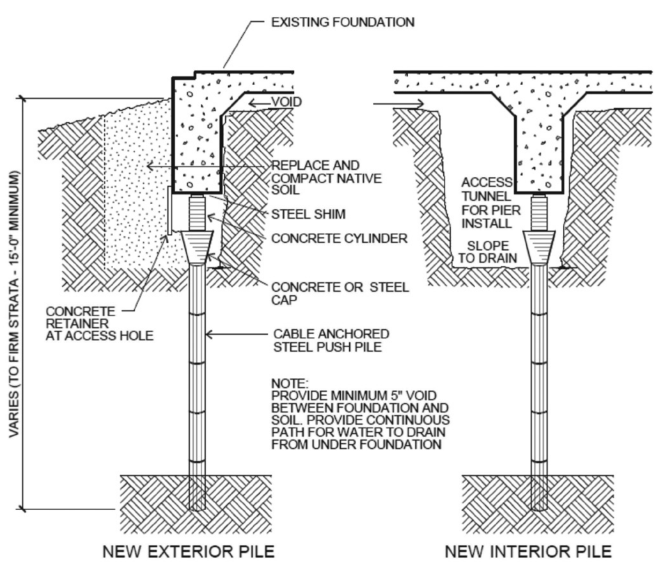


Fig. 6—Typical cross section of foundation and piling.

raised to a level position by progressively engaging 10 to 20 pilings with hand-activated bottle jacks. When the foundation is in a level position, it is secured by inserting concrete cylinders between the top of the piling and the foundation.

The exterior pilings are spaced around the perimeter at 8 to 12 ft (24.4 to 36.6 m) on center and the interior pilings are located at the intersections of the interior beams (Fig. 4). The interior pilings are installed by tunneling along the interior beams (Fig. 4 and 10). Foundations are leveled and lifted to create a void between the soil and foundation (Fig. 9). This void is usually at least 3 in. (76 mm) at the high end and may be 8 to 12 in. (200 to 300 mm) at the low end of the foundation.

Fig. 7—Hydraulic ram for installing piles.

ANALYSIS

A conventional analytical analysis and the Finite Element Method (FEM) are two methods to analyze foundations in an elevated condition. The analytical method uses a working stress method to analyze the slab, interior beams, and exterior beams separately. It is important to model the slab in a fixed condition at all four edges and consider the two-way action (Fig. 11). Moment coefficients provide a reasonably accurate method for obtaining these moments. Obviously, shear is not a concern for the slab. A moment coefficient of $WL^2/10$ can be used to estimate the beam moments. The full cross section, centerline to centerline, of the beam and slab is used when calculating the center of gravity of the concrete (CGC) for the beam analysis. Otherwise, the moments due to eccentricity of the post-tensioning will be significantly incorrect (Fig. 12).

Fig. 8—Typical exterior piling.

Fig. 9—Support at interior piling. Note void between soil and foundation after leveling foundation.

FEM analyzes the whole foundation as one complete unit (Fig. 13 through 15). ADAPT, FLOOR-PRO FEM, or similar programs can be used to analyze the foundation.

Analysis using both of these methods shows that the foundation has more than adequate strength to function in an elevated condition. Due to their significant depth, the beams have excess capacity even though they are lightly reinforced.

CODE COMPLIANCE

The original design of a post-tensioned slab-on-ground residential foundation is governed by PTI DC10.5;

Fig. 10—View of interior tunnel.

318-136 ACI STANDARD — BUILDING CODE

METHOD 3—TABLE 4—RATIO OF LOAD w IN A and SHEAR IN SLAB AND LOAD ON SUPP

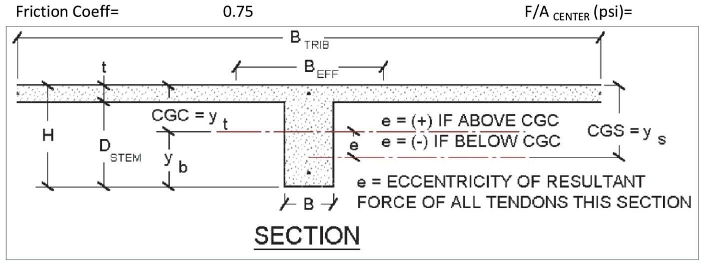
Ratio $m = \frac{A}{B}$		Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
1.00	W_A	0.50	0.50	0.17	0.50	0.83	0.71
	W_B	0.50	0.50	0.83	0.50	0.17	0.29
0.95	W_A	0.55	0.55	0.20	0.55	0.86	0.75
	W_B	0.45	0.45	0.80	0.45	0.14	0.25
0.90	W_A	0.60	0.60	0.23	0.60	0.88	0.79
	W_B	0.40	0.40	0.77	0.40	0.12	0.21

Fig. 11—ACI table for slab moments. Note: Use Case 2.

P-T Interior Beam Analysis			"As Built"	Page 1 of 2
Project:	Date:	6/13/2017	Rectangle A	

Slab/Beam/Rectangle input:		**Effective Section Properties:**				
f`c (psi) =	3000.00		Slab	Beam		Totals
Long Direction L_{SLAB} (ft) =	42.00	A (in^2)=	1,152.00	720.00	ΣA (in^2)=	1,872.00
Width Direction W_{SLAB} (ft) =	24.00	Y (in)=	2.00	14.00		
Average t_{SLAB} (in) =	4	Ay (in^3)=	2,304.00	10,080.00	ΣAy (in^3)=	12,384.00
B = Average W_{STEM} (in) =	12	Ay^2 (in^4)=	4,608.00	141,120.00	ΣAy^2 (in^4)=	145,728.00
Average D_{STEM} (in) =	20	I_o(in^4)=	1,536.00	24,000.00	ΣI_o(in^4)=	25,536.00
# L_{BEAMS} TOTAL =	3	Ay^2+I_o(in^4)=	6,144.00	165,120.00	ΣAy^2+I_o(in^4)=	171,264.00
# W_{BEAMS} TOTAL =	4	CGC=Y_t (in)=	6.615		I (in^4) =	89225.000
Max Beam Span f/f (ft)=	12.00	CGS=Y_s (in)=	4.717		S_t(in^3) =	13,488
Tansverse Span 1 f/f (ft)=	11.00	e (in)=	1.899		S_b(in^3) =	5,132
Tansverse Span 2 f/f (ft)=	11.00	Y_b (in)=	17.385		F/A $_{EDGE}$ (psi)=	263.88
Tendon Diameter (in)=	0.5	Eq. t_{SLAB} (in)=	8.167		F/A $_{DRAG}$ (psi)=	-16.41
*F_{AVE} (kips)=	28.71	Friction Coeff=	0.75		F/A $_{CENTER}$ (psi)=	247.47

# L $_{Slabs}$ Tendons TOTAL =	13
Beam Tendons EACH=	1
*Ave. Top Tendon Clear (in)=	2.00
Bottom Tendon Clear (in)=	3.00
Beam Properties:	
H (in)=	24
B_{TRIB} (ft) =	12
B_{EFF} (ft) =	12
Span to Depth Ratio=	6
Total Tendon Qty (B_{TRIB})=	7.5
Total Tendon Qty (B_{EFF})=	7.5



* From Actual Tendon Placement Information

Concentrated Loading to Beam:			Total P to 1st=	3765.51	5446.61	Page 2 of 2
Roof Loading			*1st FLOOR SLAB UNIFORM LOADING (psf)*			As Built
	Working	Ultimate		Working	Ultimate	Interior Beam
DL $_{ROOF}$ (psf)=	9.969	11.9628	DL $_{SLAB}$ (psf)=	50	60	Rectangle A
LL(psf)=	16	25.6	DL $_{ADDED}$ (psf)=	5	6	
TL (psf)=	25.969	37.5628	LL(psf)=	32	51.2	
$TRIB_{TO\ 2nd}$ (ft^2)=	144.00		TL (psf)=	87	117.2	

P_{ROOF} (lb)=	3739.54	5409.04	*BEAM UNIFORM LOADING(plf)*			*CALCULATED BEAM STRESSES AT SUPPORT*	
Second Floor Loading				Working	Ultimate	F_{TOP} (psi)=	257.0265021
	Working	Ultimate	w=	1207	1589	F_{BOTTOM} (psi)=	222.3652045
DL $_{2nd}$ (psf)=	0	0	Ultimate/ Working Ratio=		1.32		
DL $_{ADDED}$=	0	0	*ACTUAL BEAM MOMENTS (Kip*Ft)*			*ALLOWABLE CONCRETE STRESSES (psi)*	
LL(psf)=	0	0		Working	Ultimate	$ALLOW_{TENSILE}$ = -6*sqt(f`c) =	-328.63
TL (psf)=	0	0	(-)M_{BEAM}=	-23.33	-41.61	$ALLOW_{COMPRESS}$ = .45*f`c =	1350
$TRIB_{TO\ 1st}$ (ft^2)=	144.00		(+)M_{BEAM}=	18.39401	35.11	(-) = tension & (+) = compression)	
						| ALLOWTENSILE | > | fBOTTOM	
P_{2nd} (lb)=	0	0	M_{FROM}e=	34.07		|...	Stresses OK

Calculate Allowable Beam Moments

Ultimate Positive Moment		Tendon Info	**Ultimate Negative Moment**	
n_{bottom}=	1	=Tendon Qty per Beam Width at Tension Face	n_{top}=	7.5
B_{EFF}(ft)=	12	=Eff Flg or Beam Width at Tension Face	B(ft)=	1.00
f_{SE}(ksi)=	188	=Effective PT Stress after Losses= F_{AVE}/(0.153)	f_{SE}(ksi)=	188
A_{PS}(in^2/ft)=	0.153	=Tendon XSect. Area per Foot= (0.153)*(n)	A_{PS}(in^2/ft)=	1.1475
d_P(in)=	20.75	=Depth to Tendon Centroid= (h)-(dia)/2-clear	d_P(in)=	21.75
ρ_P=	0.000051	=Ratio of Prestressed Reinf=(A_{ps})/((b)*(dp))	ρ_P=	0.00440
f_{PS}(psi)=	248000	=PT Stress @ Nom Strength per ACI 318-99 eqs. (18-4 or-5)	f_{PS}(psi)=	204824
		Results		
a(in)=	0.1033	= (f_{ps})*(A_{ps})/(0.85*(f'c)*(b))	a(in)=	7.6809
d_{EFF}(ft)=	20.6983	= (d_p)-(a)/2	d_{EFF}(ft)=	17.90955882
M_u(kip*ft)=	58.90	= (0.9*(A ps)*(f ps)*(d eff))/12000	M_u(kip*ft)=	-315.70
Beam is OK			Beam is OK	

Fig. 12—Spreadsheet for analytical method.

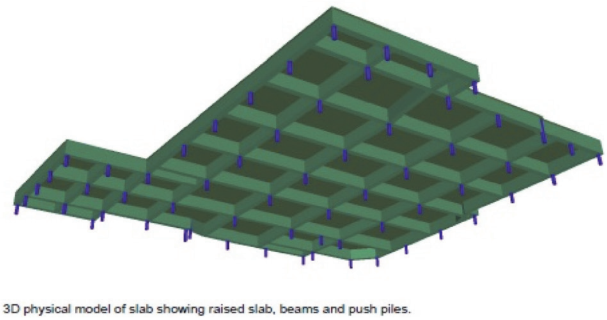


Fig. 13—ADAPT FEM.

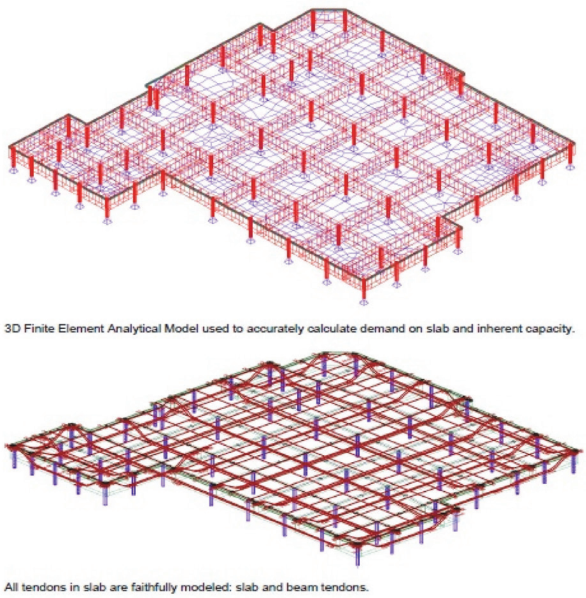


Fig. 14—View of FEM support conditions.

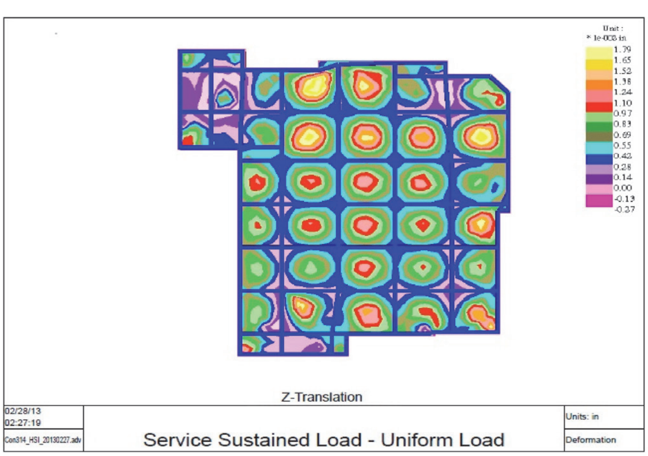


Fig. 15—FEM slab moments.

ACI 318 is not applicable to post-tensioned slab-on-ground residential foundations.

The support system has completely changed from the slab-on-ground to a pile-supported system. The serviceability requirements in ACI 318[1] are not applicable because the analysis is of an existing structure and not the design of a new structure. For existing structures, the applicable codes—ACI 318-14 Chapter 27, ACI 437,[2] and IBC 1708.1[3]—note that the analysis should be for ***strength*** and not design code compliance. ACI 437 states *Engineering judgment is critical in the strength evaluation of reinforced concrete buildings. Judgment of qualified structural engineers may take precedence over compliance with code provisions or formulas for analyses that may not be applicable to the case studies*. Because of the aforementioned documents, the minimum reinforcing bar and minimum prestressed levels are not applicable as long as the strength requirements are satisfied.

LOAD TESTS

In addition to analytical analysis, load tests have been conducted to demonstrate that the repaired foundation can carry its prescribed loads and that it meets the requirements of the building code. The testing was done in accordance with ACI 318-99, Chapter 20, and ACI 437R. The load test required the foundation to be able to carry the full dead load and full live load, including an additional load for the safety factors. The resulting test load to accomplish this was an additional 75 lb/ft^2 (3.6 kPa) over the entire bay (Fig. 16). The garage bays of a single-family residence were loaded with 3.5 x 4 ft (1.1 x 1.2 m) plywood boxes filled with water. Four boxes per bay were used so that the top of the slab could be monitored for cracking and so that instrumentation of the foundation could be done at midspan.

Calculations showed that 22 in. (560 mm) of water would be required to produce the same load that 75 lb/ft^2 (3.6 kPa) would produce. The code test procedure required that the foundation be loaded and measured at four equal increments. The report from the engineering laboratory shows the results of the test (Fig. 18). The calculated maximum deflection was 0.125 in. (3 mm) and the actual measured maximum deflection with 22 in. (560 mm) of water was 0.054 in. (1.4 mm). After the code-prescribed loading was completed, the boxes were filled to the top with 30 in. (760 mm) of water, which produced an equivalent load of 102 lb/ft^2 (4.9 kPa) with a maximum measured deflection of 0.056 in. (1.4 mm). The results show that the foundation in a pier-supported condition meets the requirements of the

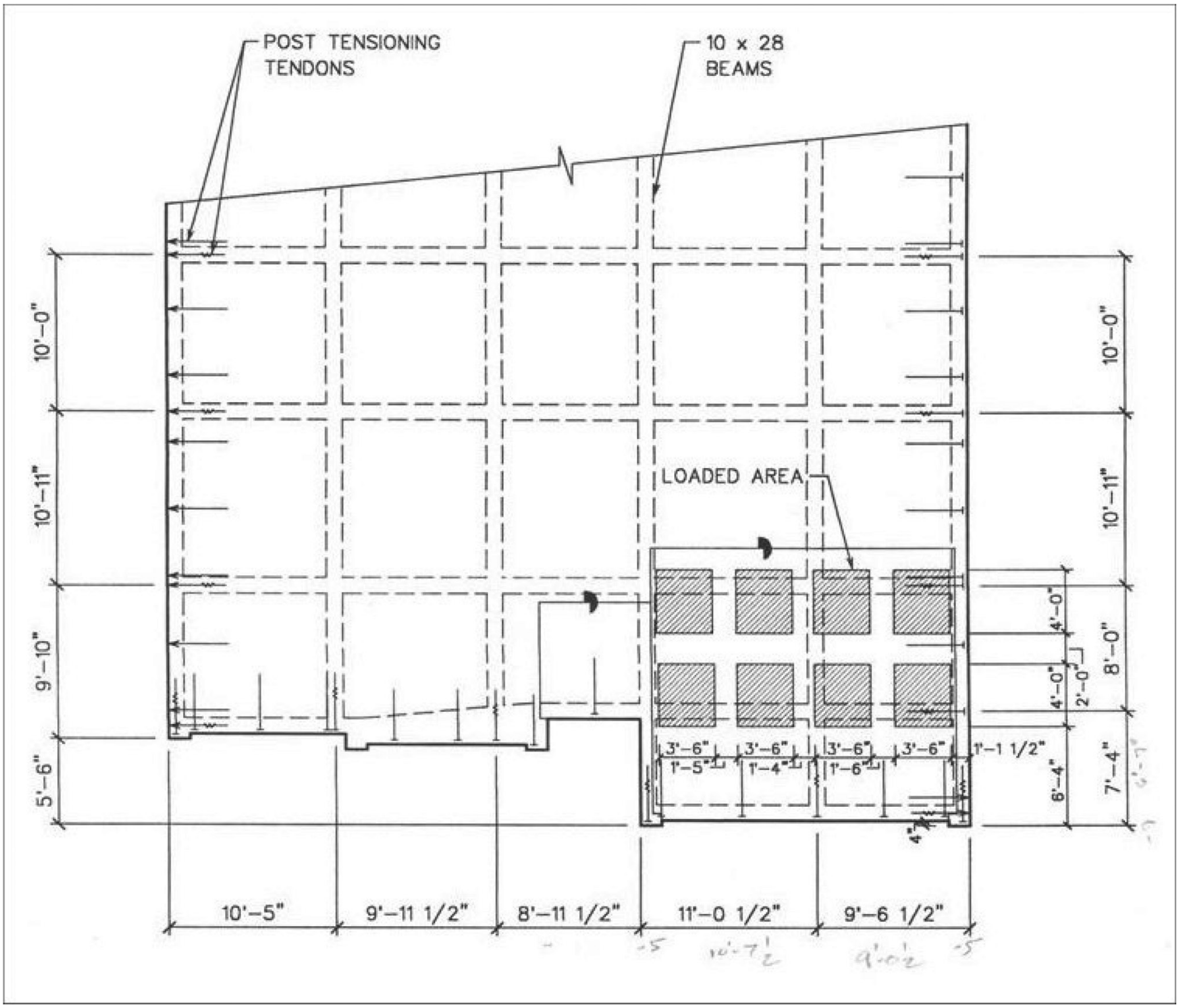


Fig. 16—Plan of load boxes from load test.

building code and has significantly greater capacity than required by code.

The pilings were also load-tested by placing a load cell on top of three in-place pilings. The load was applied individually until each piling moved (Fig. 19). The three piles tested all had a capacity in excess of 45,000 lb (200 kN). Normally, the pilings come with a lifetime warranty against settlement.

SUMMARY AND CONCLUSIONS

This repair process has been successfully implemented more than 15,000 times in the last 20 years and there are no known structural problems with the foundation in the suspended condition. The calculations, load test, and successful track record show the converted foundation

Fig. 17—Load test photos.

FLOOR LOAD TEST AT
RONE JOB NO. 00-3874

Date	West half load (inches of water)	East half load (inches of water)	Gauge 1 disp (inches)	Gauge 2 disp (inches)	Gauge 3 disp (inches)	Gauge 4 disp (inches)	Gauge 5 disp (inches)	Gauge 7 disp (inches)	Gauge 8 disp (inches)	Gauge 9 disp (inches)
07/19/2000	0	0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
07/19/2000	5.5	0	0.001	-0.001	-0.001	-0.002	-0.002	-0.001	0.000	-0.001
07/19/2000	11	0	0.002	-0.003	-0.001	-0.001	-0.001	-0.001	0.000	-0.001
07/19/2000	16.5	0	0.003	-0.005	-0.002	0.000	-0.001	-0.001	0.000	-0.001
07/19/2000	22	0	0.003	-0.006	-0.002	0.001	-0.001	0.000	0.001	0.000
07/20/2000 (24 hr. reading)	22	0	-0.001	-0.048	-0.020	0.000	-0.027	-0.004	-0.001	-0.001
07/20/2000	22	5.5	0.001	-0.048	-0.020	0.001	-0.028	-0.005	-0.001	-0.002
07/20/2000	22	11	0.000	-0.050	-0.021	0.001	-0.028	-0.006	-0.002	-0.004
07/20/2000	22	16.5	0.000	-0.049	-0.021	0.002	-0.028	-0.006	-0.002	-0.006
07/20/2000	22	22	0.001	-0.050	-0.021	0.003	-0.027	-0.007	-0.004	-0.007
07/21/2000 (24 hr. reading)	22	22	0.002	-0.052	-0.030	0.004	-0.038	-0.010	-0.008	-0.012
07/21/2000	26	26	0.003	-0.054	-0.040	0.005	-0.040	-0.010	-0.008	-0.012
	30	30	0.003	-0.056	-0.042	0.006	-0.041	-0.012	-0.010	-0.012

Upward disp (+)

Downward disp (-)

Gauge 6 was not functioning properly

Fig. 18—Deflections from load test.

Fig. 19—Load test of piling.

has adequate strength to perform in a suspended condition. Finally, the repaired foundation is better than the original foundation because it is no longer in contact with the soil.

REFERENCES

1. ACI Committee 318, "Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary," American Concrete Institute, Farmington Hills, MI, 2014, 520 pp.

2. ACI Committee 437, "Building Code Requirements for Structural Concrete (ACI 437R-03) and Commentary (ACI 318R-14)," American Concrete Institute, Farmington Hills, MI, 2003, 28 pp.

3. "2015 International Building Code," International Code Council, Country Club Hill, IL, 2015, 736 pp.

Don Illingworth *has a BSCE from the University of Wyoming and started his engineering career (after the Army Corps of Engineers) in Denver working for a consulting engineer in 1969. His first introduction to PT came in 1971 when Atlas Prestressing redesigned a precast office building into a cast-in-place PT frame. Don joined Atlas Prestressing in 1972 and had a lot of fun selling PT in a precast town. This led to involvement in a wide variety of projects such as segmental bridges in Vail, Co; Wyoming coal silos and introducing PT slab-on-ground to the Denver apartment market. In 1978, he moved to the Dallas area as the Central Division Manager for VSL. While at VSL, Don participated in FIP and VSL Symposiums in London, Stockholm, and Zermat. Don has been involved in slab-on-ground since the late 1970s and was Chair of PTI Committee DC-10, Slab-on-Ground, and was President of PTI from 1987-88. He started his own consulting business in 1988 and his involvement in PTI and DC-10 has continued. His current work involves a mixture of new designs and forensic investigations. Much of the forensic work involves investigation of residential slab-on-ground foundations. Recently, Don was a member of the PTI Executive Committee and is currently a member of the PTI Technical Advisory Board. Don is a licensed professional engineer in Texas, New Mexico, Colorado, Oklahoma, and Florida. Don is a PTI Fellow and Legend.*