COMAL COUNTY JAIL:
A CASE STUDY IN VALUE ENGINEERING USING POST-TENSIONING

POST-TENSIONING INSTITUTE ®

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Fig. 1—Rendering of Comal County Jail.

In November 2015, Comal County, TX, approved a $76 million bond package for a 150,000 ft^2 (13,900 m^2) expansion of a county jail facility in New Braunfels, TX. The project consisted of exterior tilt-up wall panels, roughly 25 ft (7.6 m) tall interior concrete masonry unit (CMU) walls, and interior structural steel columns. The geotechnical report identified a problem of active soils on the site with a potential vertical rise (PVR) of 6.25 in. (160 mm). Due to the volatile soils, a slab-on-ground foundation was impractical. Therefore, the original foundation was designed as a suspended 10 in. (250 mm) post-tensioned (PT) slab on 12 in. (305 mm) carton forms supported by concrete belled piers. The price of the project came back over budget and was at risk of being canceled altogether. The lead architect approached the engineering specialist to consult on suspended PT and to evaluate the cost-saving ideas while still using a suspended PT system.

The Engineer performed a cost-analysis of the different pier sizes, pier spacings, and slab thicknesses to develop a more economical slab design. The analysis was performed using a matrix to determine pier spacings required for each slab thickness along with loads on each pier. In the end, the Engineer came up with two options, the first being a standard 10 in. (250 mm) PT slab on 12 in. (305 mm) void boxes but with more economical pier spacings and optimization of the concrete belled piers. The Engineer recognized, along with the Architect, that the savings, although significant, were not to the extent required. Therefore, the Engineer looked at the second option, which was a substantially thinner slab with much tighter pier spacings. This design led to the use of helical piers and a patented slab-lifting system for creating the void space under the slab without the use of void-boxes. The final design resulted in a cost savings of nearly $1 million, which allowed the project to continue. In addition, the project experienced a higher than normal rain season in which the design and processes implemented of not using void boxes and still creating a void proved beneficial to schedules and cost.

PIER SIZING

The original design consisted of concrete belled piers throughout. The pier sizes ranged from 18 to 36 in. (460 to 915 mm) diameter with bells of 36 to 90 in. (915 to 2290 mm). The first step to the value engineering was to determine the load-carrying capacity of each pier along with the cost for the concrete and labor for each pier. An additional factor was considered for the time and cost to change out the drill rig with the various pier and bell sizes. Then, various spacings of piers were considered, knowing that the pier spacing would impact the slab thickness. To determine an approximate deadload weight of the slab on the piers, an estimated slab thickness was calculated. Based on this data, it was realized that using additional piers of a consistent size would be less expensive and take no more time than using the range of variable piers originally designed. The analysis of the pier sizes and their bearing capacities, along with concrete volumes, are shown in Table 1.

Looking a little closer at this analysis, if comparing a 24 in./60 in. (610 mm/1520 mm) belled pier to an 18 in./45 in. (460 mm/1140 mm) belled pier, the 24 in. (610 mm) pier has a 1.84× greater volume, but only a 1.77× greater bearing capacity. So, under some conditions, using more smaller piers can be more cost-effective than using fewer large piers. The next step was to look at each slab area and determine which pier spacing and sizes would be most cost-effective. As an example (Table 2), for Area 1 which was the first portion to be placed, a 235 psf (11.3 kPa) uniform load was assumed to determine possible pier spacings based upon the load requirements.

The original design for this area specified thirteen (13) 24 in. (610 mm) diameter piers and ten (10) 36 in. (915 mm) piers, for a total of 23 piers and an equivalent area of 111.5 ft^2 (10.4 m^2).

From Table 3, it can be seen that using thirty (30) 18 in./45 in. (460 mm/1140 mm) belled piers would be less than half the cost of the original 23-pier design.

Table 1—Drilled bell piers. Note: 1 ft. = 0.348 m, 1 k = 4.45 kN, 1 ft^3 = 0.028 m^3.

Allowable Bearing Pressures =	**18.75** ksf		
Minimum Bell to Shaft Ratio =	2.0		
Maximum Bell to Shaft Ratio =	2.5		
Pier Length =	45.0 ft	FFE = 672	Bot Pier = 635

Shaft (in)	Bell (in)	Area (sq ft)	Soil Bearing (k)	Allowable Bearing (k)	Allowable Bearing 90%(k)	Shaft Volume (cu ft)	Total Volume (cu ft)	Pier Self Wt kips
18	36	7.07	132.5	121	97	80	84	11.9
18	**45**	11.04	207.1	**195**	**164**	80	87	11.9
24	**48**	12.57	235.6	**214**	**172**	141	152	21.2
24	54	15.90	298.2	277	228	141	156	21.2
24	**60**	19.63	368.2	**347**	**291**	141	160	21.2
30	60	19.63	368.2	335	268	221	242	33.1
30	66	23.76	445.5	412	338	221	248	33.1
30	75	30.68	575.2	542	455	221	257	33.1
36	72	28.27	530.1	482	386	318	355	47.7
36	78	33.18	622.2	574	469	318	363	47.7
36	84	38.48	721.6	674	559	318	371	47.7
36	90	44.18	828.3	781	655	318	380	47.7

Table 2—Possible pier spacings based on load requirements for Area 1. Note: 1 ft = 0.3048 m; 1 kip = 4.45 kN.

	Five (5) piers @ 29 ft on center	Six (6) piers @ 23.2 ft on center
Four (4) piers @ 35 ft on center	1015 ft^2 = 239 kip = (20) piers	812 ft^2 = 191 kip = (24) piers
Five (5) piers @ 26.25 ft on center	761 ft^2 = 179 kip = (25) piers	609 ft^2 = 143 kip = (30) piers

Table 3—Possible pier spacings using belled piers based on load requirements for Area 1. Note: 1 ft = 0.3048 m.

	Five (5) piers @ 29 ft on center	Sixb(6) piers @ 29 ft on center
Four (4) piers @ 35 ft on center	(20) 24 in./60 in. piers = 62.8 ft^2	(24) 24 in./54 in. piers = 75.4 ft^2
Five (5) piers @ 26.25 ft on center	(25) 24 in./48 in. piers = 78.5 ft^2	(30) 18 in./45 in. piers = 53 ft^2

SLAB DESIGN

The original slab design was a 10 in. (250 mm) thick slab spanning between 18 and 24 in. (460 and 610 mm) belled piers. The slab needed to support stacked modular jail cell units producing point loads of 4.5 to 10 kip (20 to 44 kN), as well as 16 to 27 ft (4.9 to 8.2 m) tall CMU walls. Special considerations had to be made for these heavy concentrated loads. Furthermore, slab penetrations had to be coordinated to account for all of the plumbing and electrical lines to support the facility use. The redesign was performed using computer modeling of all the point loads, line loads, slab depressions, and slab openings. The various pier spacings were considered to determine the best cost ratio between slab thickness and pier spacing/size/depth.

The new foundation design was developed which maintained the original 10 in. (250 mm) slab thickness but was able to reduce the cost of the piers. Unfortunately, this cost savings was still not enough to save the job. Modifying the pier sizes and spacings helped reduce the cost considerably, but not enough. The controlling cost was still the 10 in. (250 mm) thick PT slab. Based on the previous analysis performed, it was noted that a thinner slab would reduce the volume of concrete and overall dead load, but it would also require a much tighter pier spacings. The thinnest PT slab which would still support the required loads was found to be 5 in. (130 mm) but with piers at roughly 13 ft (4.0 m) spacing. Sticking with concrete piers, belled or not, would make that impractical. But the closer spacing reduced the loads per pier, to which other pier options became available. All of this led to the consideration of using helical piles.

The final 5 in. (130 mm) PT slab design used 4000 psi (34 MPa) concrete and used over 1000 individual tendons for a total of 26 miles (42 km) for the entire project. Due to the lengths of each building, the longest cable was 233 ft (71 m) long and stressed at each end while a jacking gap was left for the adjacent sections. In certain areas, it was not possible to stress from the ends of the slab, so an inline coupler (dog-bone) was used to stress from the middle of the slab.

The basic design for each PT slab (Fig. 2) consisted of four banded tendons running along the piers in the long pier span direction and two distributed tendons running along the piers in the short pier span direction. Between these distributed tendons, individual distributed tendons were spaced no more than 3 ft (0.91 m) on center. In the other direction, between the banded cables, shrinkage cables were spaced no more than 3 ft (0.91 m) on center. The precompression ratio for the slabs averaged 281 to 459 psi (1.9 to 3.2 MPa).

With the use of computer modeling, the entire slab for each section was modeled based on the aforementioned criteria to confirm the structure met all design requirements. One of the important details within the section shown in Fig. 3 is the tilt-wall section that created a courtyard and having to work the PT design and implementation of a beam and piers under the tilt-wall itself. The finished project could also be used to compare against another computer model (Fig. 3) to ensure all features had been implemented.

HELICAL PILES

Helical piles (Fig. 4) are manufactured steel modular products that consist of a series of helixes on the lead section and extensions. According to the product's website, experience has shown that corrosion of the galvanized helical piles and anchors has not been a problem. Life expectancies are typically in the 200- to 250-year range. In areas with corrosive soils, further testing is recommended. The lead section is drilled into the ground first and additional extensions are added to push the pile deeper until it reaches the proper depth and torque. In areas of expansive soil, it is essential to get the top helix below the active zone.

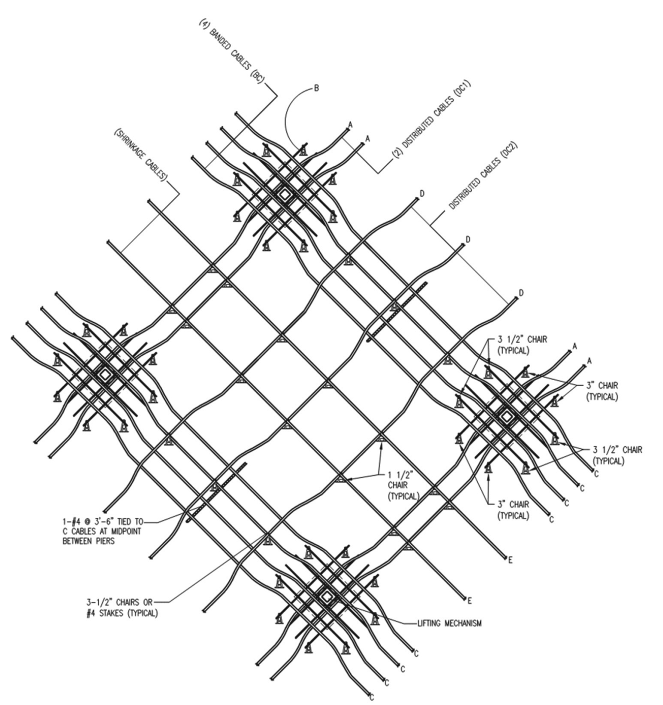


Fig. 2—Isometric of cable profiling. Note: 1 ft = 0.3048 m.

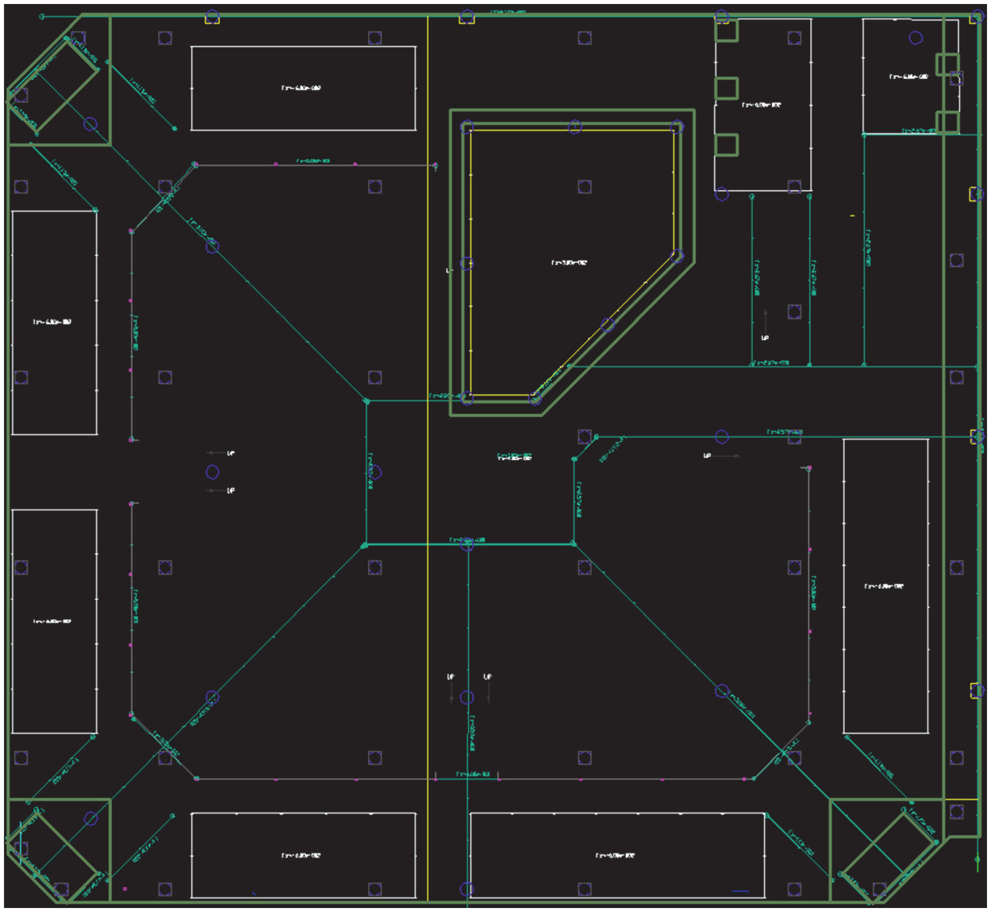


Fig. 3—Computer model for Area H.

Fig. 4—Typical square-shaft helical pier.

Furthermore, a standard torque correlation factor is used to convert an installed torque value to a design compressive load capacity.

The engineer looked at four helical pier sizes based upon the range of loads being considered (Table 4).

The 2018 IBC Section 1810.3.3.1.9 requires helical piles to have a mechanical (ultimate) load capacity that is double the design (working) load. Considering the cost per pier versus the working load capacity, the SS150 provided the most economical design to consider initially. Given that the slab needed to support heavy CMU walls and point loads from the jail cells, the larger piers could be used at strategic locations as needed.

By reducing the pier spacings to approximately 13 ft (4.0 m) on center, the slab thickness was able to be reduced to 5 in. (130 mm) within the body of the slab. The slab would be supported on 26 x 26 in. (660 x 660 mm) capitals, giving a total depth of 10 in. (250 mm). Thickened 10 in. wide x 26 in. deep (250 mm wide x 660 mm deep) "shovel beams" were used beneath the CMU walls to act as beams to limit deflections (Fig. 5). These "shovel beams" were also designed to distribute the heavy line loads and provide embedment depth for the CMU reinforcement. To limit the effect on the PT slab itself, a series of helical piles were placed under the shovel beams. The pier spacings for the shovel beams were designed at

Table 4—Helical pier sizes based on load. Note: 1 kip = 4.45 kN.

Helical size	Working load, kip	Ultimate load, kip
1.5 in. square shaft (SS150)	35	70
2-7/8 in. round shaft (RS2875.276)	36	72
1.75 in. square shaft (SS175)	52.5	105
2.0 in. square shaft (SS200)	80	160

a closer spacing than the 13 ft (4.0 m) which was considered for the slab and this was done to keep the stresses to a minimum while maintaining the same depth.

The only issue with this design is the thinner slab and helical piers could not handle the superstructure column loads nor the tilt-wall panels as the loads were extremely high. Therefore, a modified design was considered where the superstructure columns and

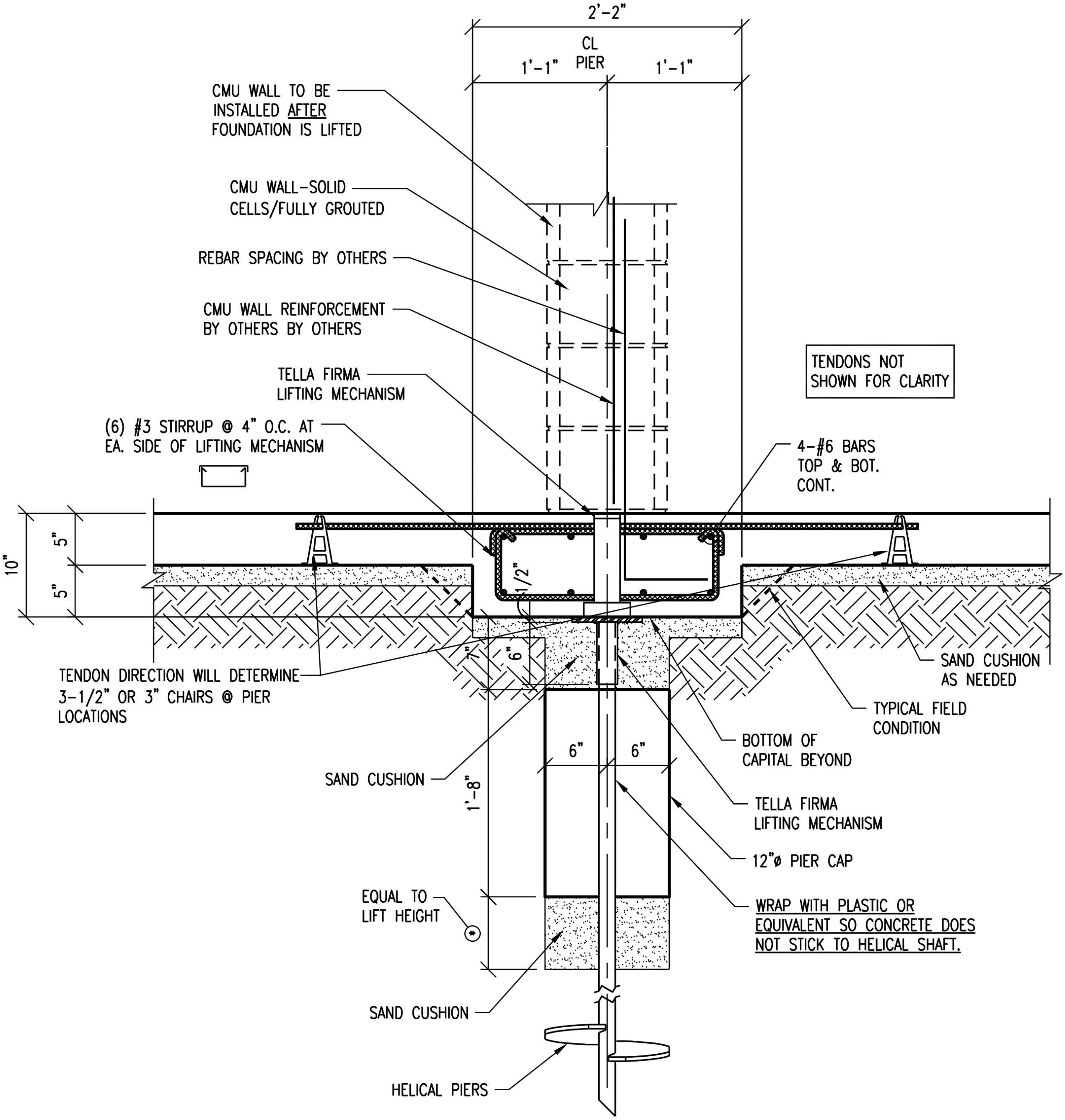


Fig. 5—Detail of shovel beam with CMU.

Fig. 6—Installation of helical piers.

Fig. 7—Lifting mechanism.

tilt-up walls were still supported by concrete belled piers, while the slab encompassed those elements and supported the CMU walls. The PT slab was supported by nearly 1600 helical piles (Fig. 6). Furthermore, by supporting the slab with tighter pier spacings, this also reduced the load on the superstructure piers, which allowed them to be smaller, further reducing the project cost.

NO VOID BOXES

The 6.25 in. (160 mm) PVR identified by the geotechnical engineer required the original design to have a 12 in. (305 mm) void space. Typically, these voids are formed by carton forms that are designed to temporarily support the slab while it is being formed and cured, but over time, the forms are intended to absorb water and deteriorate, leaving a gap between the active soils and the slab. This gap allows the soil to expand and contract without transferring load to the slab.

With the foundation design changing from a 10 in. (250 mm) slab with belled piers at 30 ft (9.1 m) on center to a 5 in. (130 mm) slab with helical piles at 13 ft (4.0 m) on center, the Engineer was able to consider another alternative for creating the required void space which the Engineer has used many times before.

The engineer suggested using a patented slab-lifting technology that uses steel mechanisms that are set on the piers and embedded into the slab which is formed directly on the ground (Fig. 7). After the slab is cured and PT tendons are stressed, threaded bolts are inserted into the mechanism and manually turned to lift the slab, which creates the void below the slab.

Using this process eliminated the need for carton forms on the slab portion. Carton forms were still used in isolated locations such as pour strips and leave-outs after the slab was lifted, but the primary slab placements were completed without the need for void-boxes. This was a critical concept for the General Contractor due to their concern with void-boxes and rain. Oftentimes when using carton forms, especially with large slab areas like this project had, contractors have had to schedule their slab make-up around expected weather conditions. It could take several days, if not weeks, for a crew to set out the carton forms, place tendons and reinforcing bar, and finally start placing concrete. If it rains during that time, all of the carton forms have to be replaced and all of the reinforcement reset. This causes construction delays and increases project costs. Additionally, during initial project scheduling, the slab make-up portion was expected to occur in the fall and spring when rain days are most frequent in New Braunfels.

With the potential for rain delays during construction and the thickness of the original slab design, the general contractor planned on placing the original slab design in 10 separate placements. By reducing the slab thickness (and the total volume of concrete needed) and eliminating the carton forms, the new design was able to be completed in seven placements.

SPECIAL DESIGN CONSIDERATIONS

The thickened slab beam for the CMU walls was mentioned previously, but additional design challenges had to be considered. Because the slab was designed as a PT slab, the cables needed to be accessible for tensioning after the concrete was placed and cured. With tilt-up walls designed along the perimeter of the structure and between buildings, a 3.5 ft (1.1 m) pour strip was provided between the edge of slab and the tilt-up panels (Fig. 8). This pour strip allowed access to the jacking points. After the slab was lifted, reinforcing bar dowels were embedded into the slab and wall panels, and carton forms were used to support the pour strip.

Between buildings, corridor walkways were used in the design, but the space was too narrow to justify additional helical piers and lifting mechanisms. In these locations, small corbels were detailed as extensions of the adjacent slabs so that an additional carton form-supported slab could be placed between the CMU walls (Fig. 9).

Another unique design element was a 1 ft (0.30 m) raised platform intended for temporary holding cells with CMU walls. An extra foot of slab thickness would not have worked with the lifting mechanism, so the Engineer

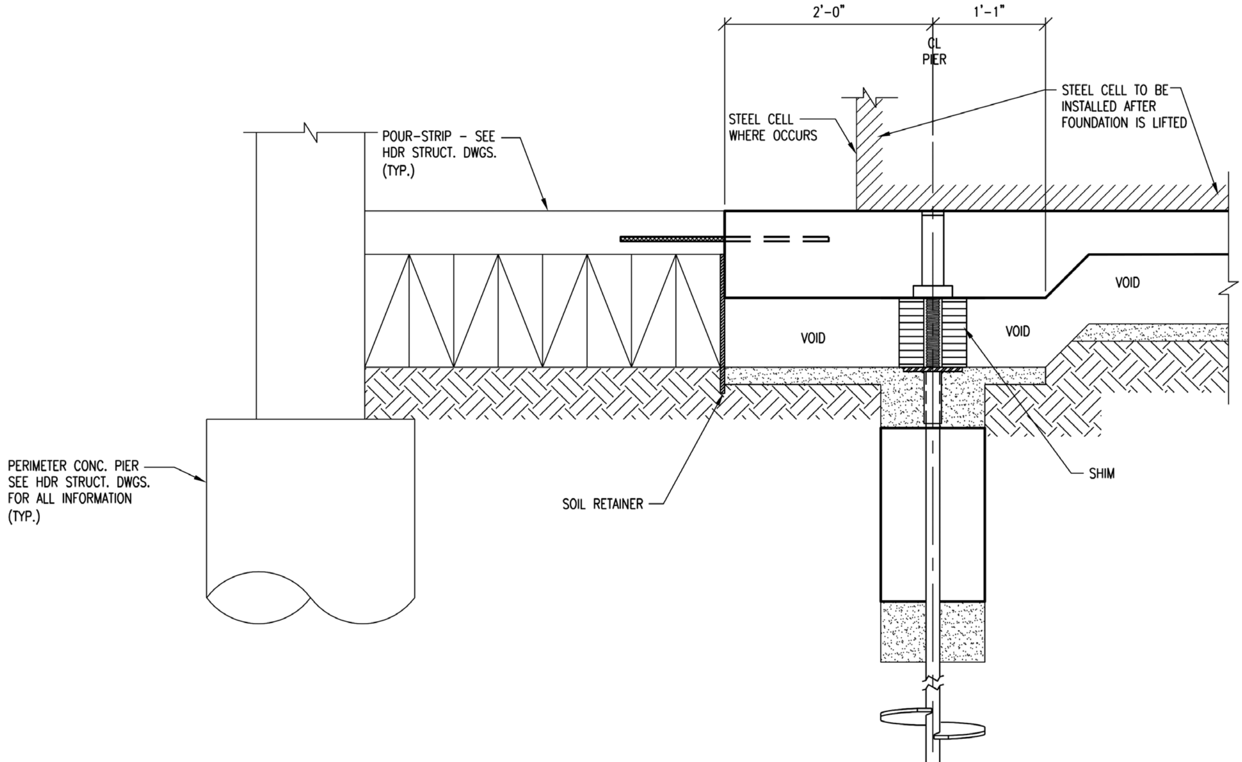


Fig. 8—Leave-out detail at tilt-wall. Note: 1 ft = 0.3048 m.

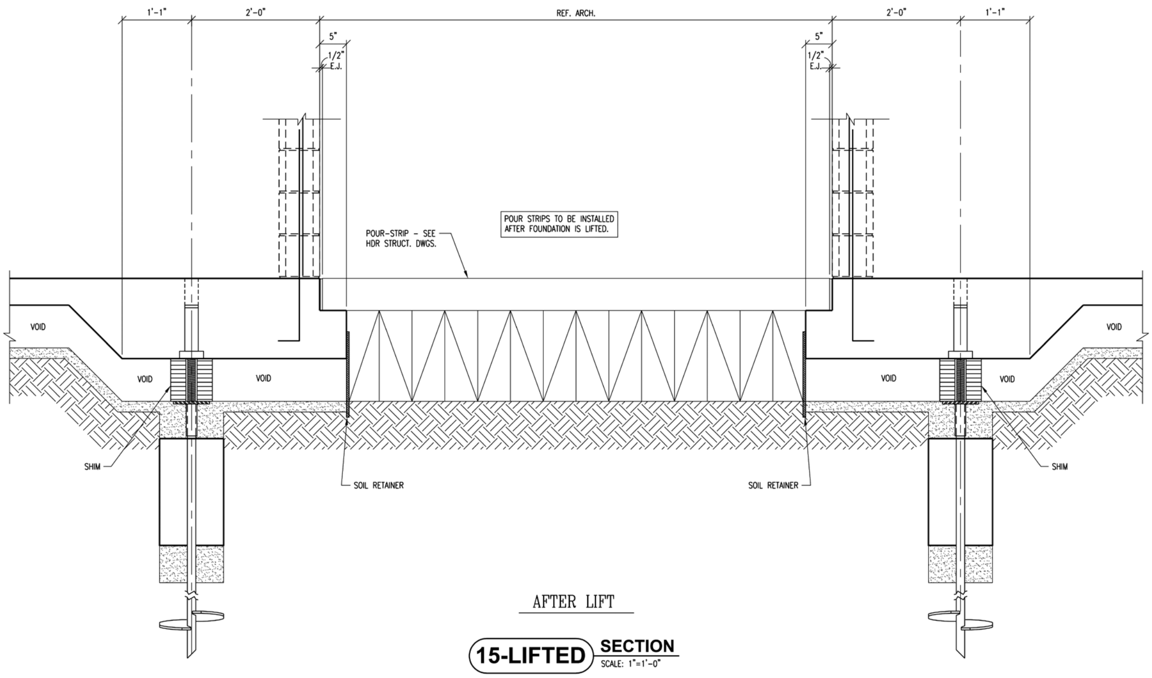


Fig. 9—Leave-out between buildings. Note: 1 ft = 0.3048 m.

designed a "double slab" (Fig. 10). After the typical slab was raised, a level of styrofoam was placed in the designated area and a second 6 in. (150 mm) slab was placed on top.

Finally, because the foundation was being placed as a slab-on-ground and was going to be raised after the fact, polyvinyl chloride (PVC) sleeves were used over the plumbing penetrations (Fig. 11). The PVC sleeves allowed the slab to be raised without effecting the plumbing. The plumbing penetrations were sealed after the slab was raised.

CONCLUSIONS

Ultimately, general contractors presented bids on both designed options. The winning bid chosen by the county was for the 5 in. (130 mm) thick PT slab on helical piles with the patented lifting process. This selection was nearly $1 million lower than the original 10 in. (250 mm) slab design. Installation of the 1600 helical piles began in May 2018 and was completed in 6 weeks. Slab makeup began soon after and was able to be performed in half the time of a thicker slab design. Plumbing and electrical utilities were placed just below grade and penetrated the slab with the use of sleeves. The sleeves allowed the slab to be lifted without effect to the plumbing/electrical pipes. The first slab was placed in October of that year, with the other placements following in sequence.

The tilt-up panels and superstructure framing was installed, and the slab was used to brace the walls until the roof diaphragm was installed. The first slab was lifted in February 2019 and the last of the seven slabs was lifted in September 2019. Full handover of the facility to the Comal County Sheriff's Department is slated for December 2020.

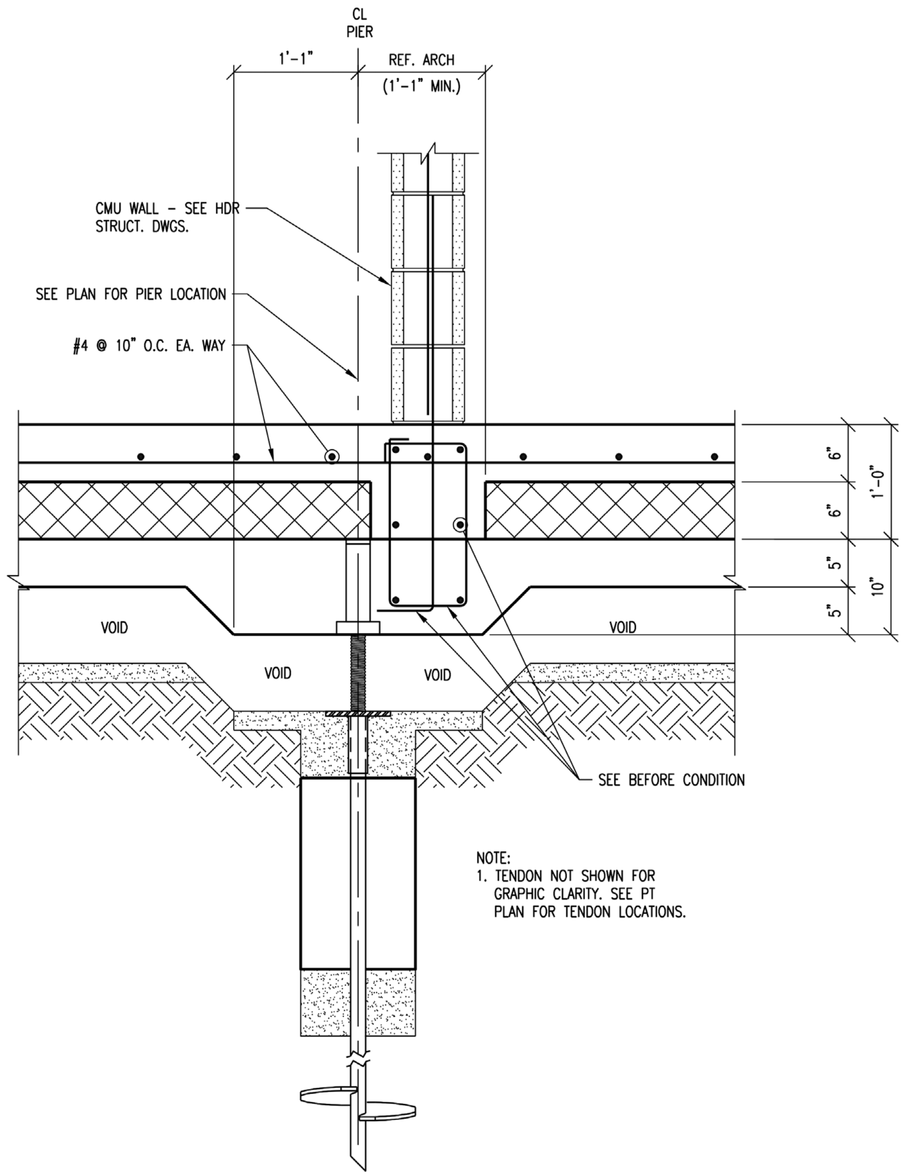
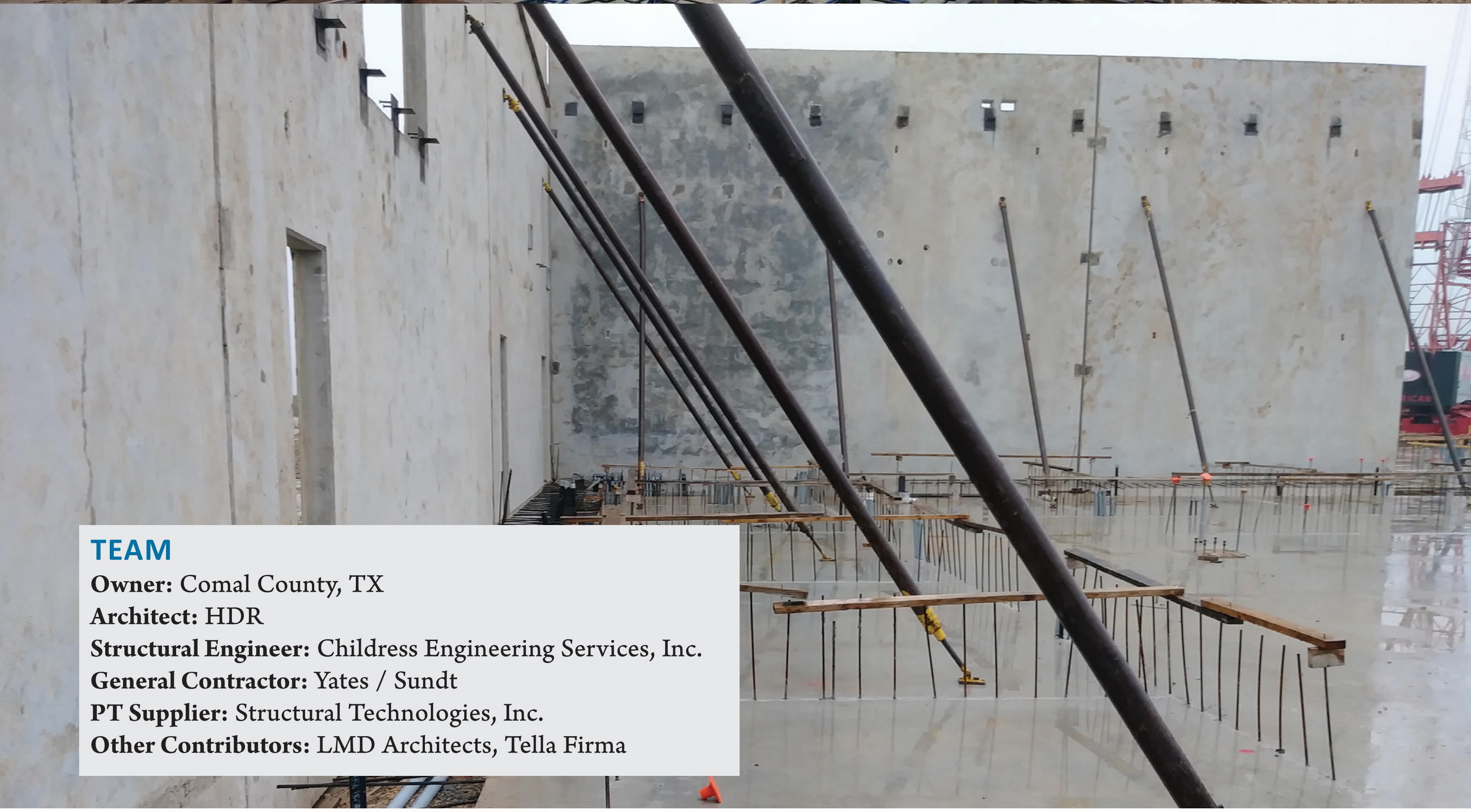


Fig. 10—Detail with double slab configuration. Note: 1 ft = 0.3048 m.

Fig. 11—Sleeves at plumbing penetrations.



TEAM

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