Report on Column Repair and Construction

THORSTENSON RESIDENCE
1569 NW 167TH STREET
SHORELINE, WASHINGTON 98177-3852

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URS Corporation
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Project: Final Report for 1569 NW 167th Street, Shoreline, Washington 98177-3852
Attn.: Mr. Bob Thorstenson
Subject: Restoring Structural Integrity of the Slab and the pool

BACKGROUND

On May 22, 2001, I visited the Thorstenson residence located at 1569 NW 167th Street, Shoreline, WA to inspect a pier underneath a slab that contained a swimming pool. The pier was structurally detached from the slab, creating a hanging slab without any vertical support.

The outdoor in-ground gunite swimming pool is located in the eastern portion of the rear yard, and the surrounding area of the pool is a slab which is partially supported on fill material in conjunction with concrete beams and columns (Figure 1).

Figure 1: The rear yard

The edge of the slab is fenced and there is a steep slope leading to the Boeing Creek. The fill underneath the slab appears to have suffered from substantial soil erosion, as is evident in Figure 2.

Figure 2: The fence, concrete slab, and the soil erosion

This report summarizes the results of engineering investigation and evaluation of the foundation support, and construction of a new pier to support the slab.

A geotechnical investigation was performed by URS Corporation of Seattle, WA to obtain information on the soil conditions and the adequacy of existing foundation support for the pool, and to provide recommendations for rehabilitation work. A site plan for the rear yard is presented on Figure 3.

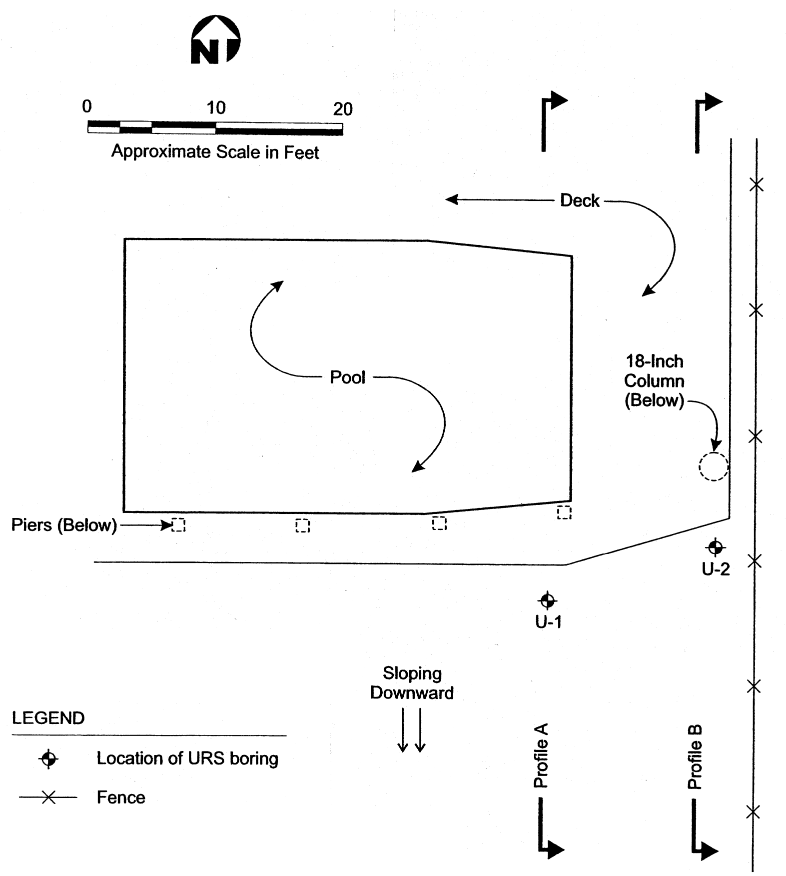

Figure 3: Site Plan

The scope of URS services during this phase of the work consisted of drilling two boreholes, conducting laboratory tests on selected soil samples to measure physical properties, investigating the stability of the slope below the pool, examining the embedment depths of existing columns on the downhill side of the pool, and providing recommendations for slope stabilization, foundation support and construction. The following is the summary of the geotechnical investigation report conducted by URS Corporation.

SITE CONDITIONS

The swimming pool and its surrounding concrete deck are situated near the top of a southern facing slope in a broad ravine occupied by Boeing Creek. The height of the most steeply inclined portion of the slope is approximately 25 to 30 feet from the backyard level. The upper half the slope is steep, with an inclination of approximately 45 degrees. The slope becomes flatter in the lower half, where the inclination is more typically 20 to 30 degrees. Near the fence along the eastern border of the property, the slope is more uniform with a moderate inclination of roughly 20 degrees overall. The slope face within about 6 to 8 feet below the pool bottom is covered by a 2 to 3 –inch thick wire mesh-reinforced concrete slab, which was apparently intended to control erosion on this steep portion of slope. The slab has been damaged and removed at some locations. The slope and slab have a thick vegetative cover of mostly ivy, with some small trees situated near the toe. Beneath the deck near the east end of the pool the ground surface is bare or sparsely vegetated.

The pool bottom and a portion of the deck appear to be supported on native or fill soil, although the pool and deck are also structurally supported with a system of concrete beams and vertical piers that transfer the loads to the foundation soil at various depths below the ground surface. From information reported by the owner, we understand that the piers are uniformly distributed beneath the pool. The spacing of the piers is not known. Direct observation indicates that the pool overhangs by about 4 feet at the location of the westernmost pier along the face of the slope and by about 3 feet at the location of the next pier to the east. Loss of contact between the top of the easternmost concrete column and the underside of the slab occurred at some unknown time in the past.

Two episodes of ground movement are reported to have occurred in the eastern part of the back yard since the construction of the house in 1958. The first event took place right after construction of the house and resulted in the downslope movement of an earlier swimming pool built on fill without intermediate foundations. The second event took place on Christmas day 1969 and consisted of a localized slope failure resulting from erosion of slope material by surface water. A concrete pier involved in the 1969 event still sits on the slope near the eastern border of the property. We understand that the source of surface water flow has been diverted since that time, and no subsequent erosion events have been reported. No changes were reported to have occurred as a result of the February 28, 2001 Nisqually Earthquake.

SUBSURFACE CONDITIONS

Subsurface soil and groundwater conditions were investigated by drilling two boreholes at the locations shown on Figure 3. Access to drilling sites was limited by low overhead space beneath the deck and steep slope conditions just below the pool itself. The boreholes were drilled using a portable gasoline-powered hollow stem auger to a depth of 15 feet each. Standard Penetration Tests (SPT) were performed using a standard split spoon sampler that was driven by an automatic hammer. Penetration resistance values (N-values in blows per foot) obtained during the sampling process were recorded for use in estimating the engineering behavior of the soils.

The field exploration was coordinated by a URS representative who located the borings, classified the materials encountered, maintained a log of each boring and obtained samples of the various strata for additional visual examination and future laboratory testing. Graphical representations of the soils encountered in the borings U-1 and U-2 are presented in logs in Figures 4 and 5 (also refer to Figure 3 for the location of borings).

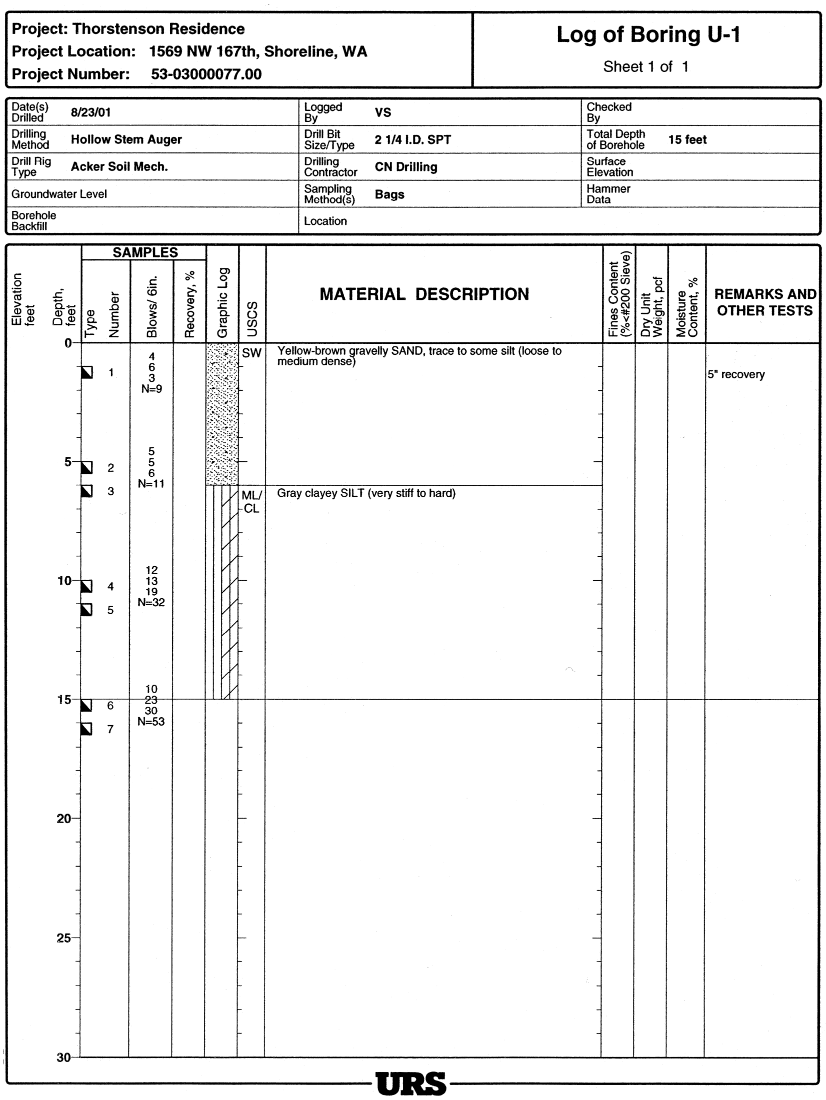

Figure 4: Log of boring U-1

Project: Thorstenson Residence **Project Location: 1569 NW 167th, Shoreline, WA** **Project Number: 53-03000077.00**	**Log of Boring U-2** Sheet 1 of 1

Date(s) Drilled	**8/23/01**	Logged By	**VS**	Checked By	
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**2 1/4 I.D. SPT**	Total Depth of Borehole	**15 feet**
Drill Rig Type	**Acker Soil Mech.**	Drilling Contractor	**CN Drilling**	Surface Elevation	
Groundwater Level		Sampling Method(s)	**Bags**	Hammer Data	
Borehole Backfill		Location			

Elevation feet	Depth, feet	SAMPLES Type	Number	Blows/ 6in.	Recovery, %	Graphic Log	USCS	MATERIAL DESCRIPTION	Fines Content (%<#200 Sieve)	Dry Unit Weight, pcf	Moisture Content, %	REMARKS AND OTHER TESTS
	0						SW	Fill: brown gravelly SAND with roots and leaves				
			1	1 1 1 N=2			SW	Brown gravelly SAND, trace of silt (loose to medium dense)				5" recovery
	5		2	3 3 4 N=7								6" recovery
							ML	Gray clayey SILT (very stiff to hard)				
	10		3 4 5	13 18 23 N=41								Oxidized red to brown vertical fissures sand lenses
	15		6 7	18 18 27 N=45								Sand lenses
	20											
	25											
	30											

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Figure 5: Log of boring U-2

The soils have been classified in general accordance with the Unified Soil Classification System that is summarized on Figure 6.

Key to Log of Boring and Descriptive Terms for Soil

Unified Soil Classification System (ASTM D2487 & D2488)

Major Divisions			Symbols: Letter	Typical Descriptions
Coarse Grained Soils — More than 50% of No. 200 Sieve Size	Gravels — More than 50% of Coarse Fraction Retained in No. 4 Sieve	Clean Gravels (less than 5% fines)	GW	Well-Graded Gravels, Gravel-Sand Mixtures, Little or no Fines
			GP	Poorly-Graded Gravels, Gravel-Sand Mixtures, Little or no Fines
		Gravels with Fines (more than 12 % fines)	GM	Silty Gravels, Gravel-Sand-Silt Mixtures
			GC	Clayey Gravels, Gravel-Sand-Clay Mixtures
	Sands — More than 50% of Coarse Fraction Passing through No. 4 Sieve	Clean Sand (less than 5% fines)	SW	Well-Graded Sands, Gravelly Sands, Little or no Fines
			SP	Poorly Graded Sands, Gravelly Sands, Little or no Fines
		Sands with Fines (more than 12 % fines)	SM	Silty Sands, Sand-Clay Mixtures
			SC	Clayey Sands, Sand-Clay Mixtures
Fine Grained Soils — More than 50% of Material is Smaller than No. 200 Sieve Size	Silts and Clays	Liquid Limit Less than 50%	ML	Inorganic Silts and very Fine Sands, Rock Flour, Silty or Clayey Fine Sands or Clayey Silts with Slight Plasticity
			CL	Inorganic Clays of Low to Medium Plasticity, Gravelly Clays, Sandy Clays, Silty Clays, Lean Clays
			OL	Organic Silts and Organic Silty Clays of Low Plasticity
	Silts and Clays	Liquid Limit Greater than 50%	MH	Inorganic Silts, Micaceous or Diatomaceous Fine Sand or Silty Soils
			CH	Inorganic Clays of High Plasticity, Fat Clays
			OH	Organic Clays of Medium to High Plasticity, Organic Silts
Highly Organic Soils			PT	Peat, Humus, Swamp Soils with High Organic Contents (see ASTM D4427-92)

Abbreviations

SA	Sieve Analysis
M	Moisture
DD	Dry Density
AL	Atterberg Limits
HA	Hydrometer Analysis
C	Consolidation
Pc	Constant Head Permeability
Pf	Falling Head Permeability
DS	Direct Shear
TX	Triaxial
TV	Torvane Shear
LV	Laboratory Vane Shear
PP	Pocket Penetrometer
OVA	Organic Vapor Analyzer
OC	Organic Content
N	Number of hammer blows for last 12 inches sampled

Relative Density or Consistency

Coarse-Grained Soils: Relative Density	N, SPT Blows / ft	Fine-Grained Soils: Relative Consistency	N, SPT Blows / ft
Very loose sand	0 - 4	Very soft	< 2
Loose	4 - 10	Soft	2 - 4
Medium dense	10 - 30	Medium stiff	4 - 8
Dense	30 - 50	Stiff	8 - 15
Very dense	Over 50	Very stiff	15 - 30
		Hard	Over 30

Sampler Symbols

- 3" O.D. Split Spoon Sample with brass rings
- 3" O.D. Shelby Tube Sample
- Core
- Piston Sample
- Non-standard penetration test
- Grab Sample
- 2" O.D. Split Spoon with 140lb Hammer and 30-inch drop (SPT)

Minor Descriptors

Trace	0 - 5%
Slightly (clayey, silty, sandy, gravelly)	5 - 12%
Clayey, silty, sandy, gravelly	12 - 30%
Very (clayey, silty, sandy, gravelly)	30 - 50%

Moisture Content

Dry	Absence of moisture, dusty
Moist	Damp but no visible water
Wet	Visible free water, from below the water table

NOTES:

1. Descriptions and stratum lines are interpretive; field descriptions may have been modified to reflect lab test results. Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced; they are not warranted to be representative of subsurface conditions at other locations or times.
2. Dual Symbols are used to indicate borderline soil classifications

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Figure 6: Key to log of boring and descriptive terms of soil

The typical soil profile consisted of a surface disturbed soil and/or fill zone, followed by a gravelly sand layer 7 to 10 feet thick, then a clayey silt layer in which the borings were terminated. A further description of these layers is presented below:

Fill

The sand and gravelly sand fill is derived from on-site native soils, and is probably associated with excavation of the basement of the house in 1958. The conditions of the fill along the length of the pool foundation appeared to be variable. The fill appeared to be loose in the upper 1 to 1-1/2 feet of the easternmost zone beneath the deck, and became more dense with increasing depth. The fill beneath the pool appeared to be dense with some degree of cementation, judging from the considerable resistance offered to hand excavations around the support piers.

Gravelly Sand

This deposit consists of loose to medium dense gravelly sands deposited in glacial lake deltas by meltwater streams during the recession of the Vashon continental ice sheet. SPT-N values varied between 7 and 11 blows per foot. The results of a sieve analysis test indicated a fines (silt and clay) content of 18 percent for that sample.

Clayey Silt

The hard clayey silt consisted of pre-Fraser nonglacial sediments, which are typically interbedded with the glacial lacustrine “Lawton” formation. Interbeds of sand stained by iron oxides were observed. SPT-N values varied between 32 and 53 blows per foot.

Groundwater

Groundwater seepage was not observed at the ground surface on the slope face, and was not encountered at any depth during drilling of the borings. It should be noted that the field exploration program was performed during the dry season.

An estimated soil profile beneath the east end of the pool is shown on Figure 7.

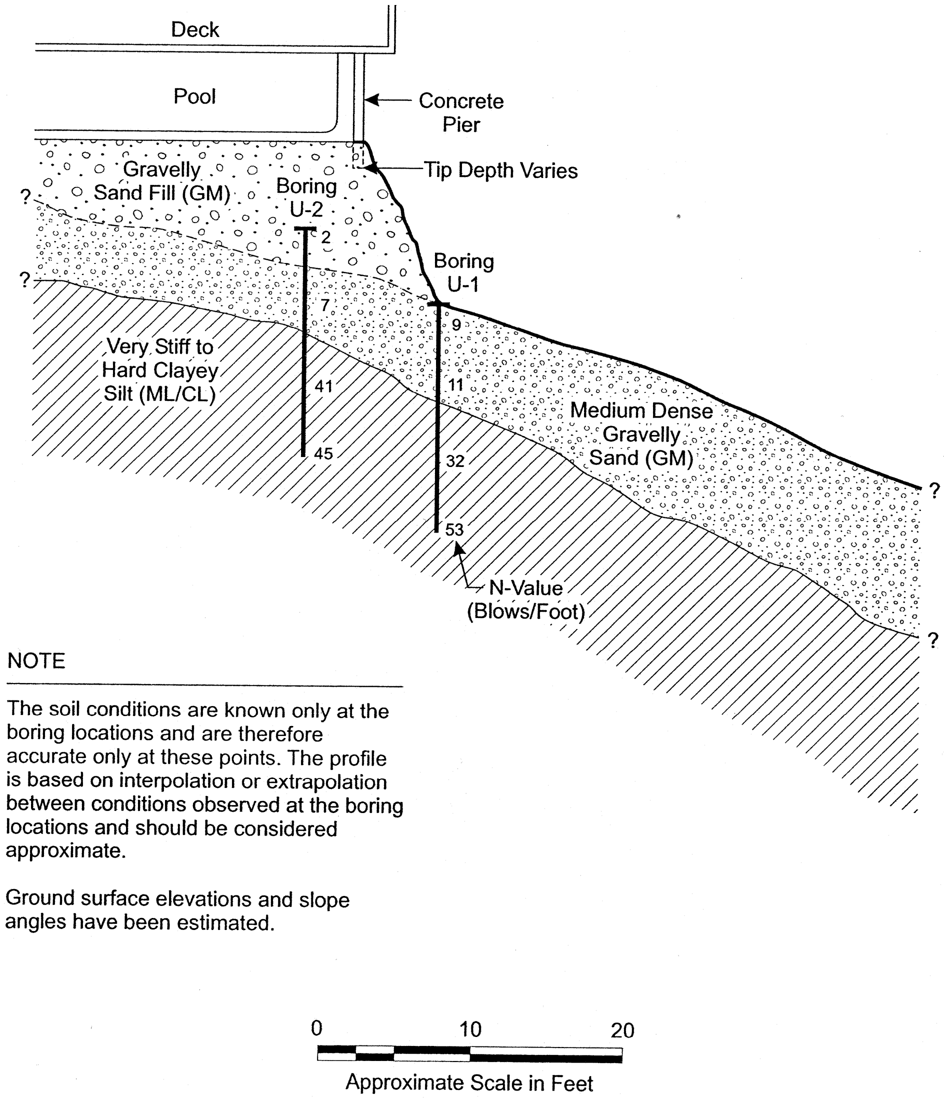

Figure 7: Soil profile beneath the east end of the pool

DESCRIPTION OF FOUNDATION INSPECTION

The swimming pool is supported by the concrete piers, most of which seem to extend through the fill soil to underlying native soil at various depths. The surrounding concrete deck is supported on the edge of the pool and on concrete beams that are in turn supported on the 18-inch diameter concrete column at the southeast corner and on the 7-inch square concrete piers located at approximately 10-feet on centers along the southern edge of the pool. It is not clear how much load these piers may be carrying. The 18-inch column is currently separated from the underside of the concrete deck. As a consequence the concrete slab is temporarily acting as a cantilever. We presume that the column directly supported the deck at one time in the past, and settled due to compression of the foundation soil or downslope movement from erosion or stress-induced instability.

The embedment depth of the 18-inch concrete column and the square piers along the southern edge of the pool was examined first by URS, then later by Sundance Construction (contractor) by hand-excavating and probing using a pointed steel rod. From the field inspection it appeared that the square pier at the southeast corner of the swimming pool and the next pier along the slope face were embedded a depth of at least six feet and eight feet, respectively. The bottoms of these two piers could not be exposed due to hard excavating conditions and/or caving of looser overlying soil. The bottom of the third easternmost pier along the slope face was reached at a depth of eleven feet beneath the bottom of the pool. The soil around the base of the 18-inch column and in this general vicinity appeared to be less competent than fill beneath the pool, and likely reflects the history of disturbance from erosion activity. The medium dense gravelly sand fill soil at the base of the pool appeared to be firm and showed no evidence of sloughing or downslope movement at the time of our visits.

DESCRIPTION OF SLOPE STABILITY ANALYSIS

Stability analyses of the soil slope beneath the pool and deck were performed by URS Corporation. Two critical profiles were analyzed:

PROFILE A - passing through the eastern-most part of the property, where past ground movement from surface water flow beneath the slab has been recorded; and

PROFILE B - passing beneath the pool itself at a location where the fill beneath the pool is thicker and steeper than elsewhere.

The geometry of the profiles was estimated based on visual observations and field measurements since topographic information was not available for this study. Static and seismic stability analyses were conducted using the commercially available computer program SLOPE/W (GEO-SLOPE International, Ltd.), which uses a search routine to calculate the minimum factor of safety for circular slip surfaces. The seismic analysis was carried out in accordance to the 1997 Uniform Building Code, using a design seismic event with a 475 year return period. The “seismic coefficient” required in the analysis was taken as one-half of the estimated Peak Ground Acceleration (PGA). A PGA value of 0.32 g has been published by the USGS (http://geohazards.cr.usgs.gov/eq) as expected for the area of Shoreline, Washington during the 475-year return period event.

Two scenarios were considered in the slope stability analysis:

SURCHARGE - The load applied to the slope due to the weight of the pool and the surrounding deck was incorporated as a uniform vertical pressure of 525 ponds per square foot;

NO SURCHARGE - No load was applied to the slope due to the weight of the pool and the surrounding deck because of the presence of the load supporting piers beneath the pool.

The second (No Surcharge) scenario is considered the most realistic based on field observations and the reported history of construction of the pool. The load applied to the slope by the eastern end of the deck via the 18-inch diameter column (assuming it was and will in the future be directly supporting the deck) was incorporated as a concentrated load of 17 kips applied at the ground surface. This assumption is conservative.

Shear strength parameters for the soil layers were selected based on published correlations with N-values in similar soils and our past experience with soils in the Puget Sound area. The fact that the sandy gravel fill beneath the pool has been standing at steep angles without major instability indicates that interlocking and light cementation, hence cohesion, must be present. This material was accordingly modeled with a friction angle of 40 degrees and a cohesion of 300 psf. Cohesion was ignored in the native gravelly sand, and a lower friction angle of 35 degrees was used to reflect the apparently somewhat less dense condition compared with the compacted fill. To account for the possible occurrence of groundwater in the wet season, the analysis was performed assuming that 2 feet of water is perched on the hard clayey silt layer.

Graphical results of the slope stability analyses at Profile A are shown on Figures 8 and 9 for the static loading case and on Figures 10 and 11 for the seismic case. The potential slide mass with the lowest factor of safety is shaded in the figures. The analyses indicated that an adequate factor of safety of 1.56 is obtained for the most realistic "No Surcharge" static loading scenario. The engineering community considers a safety factor of at least 1.5 as adequate for permanent slopes. A factor of safety of 1.26 was obtained for the less-realistic "Surcharge" static loading case. Both the "surcharge" and "No-Surcharge" scenarios resulted in adequate safety factors for the seismic case. The seismic factor of safety of 1.08 for the "Surcharge" scenario is sufficiently close to the desired 1.1 value to be considered satisfactory.

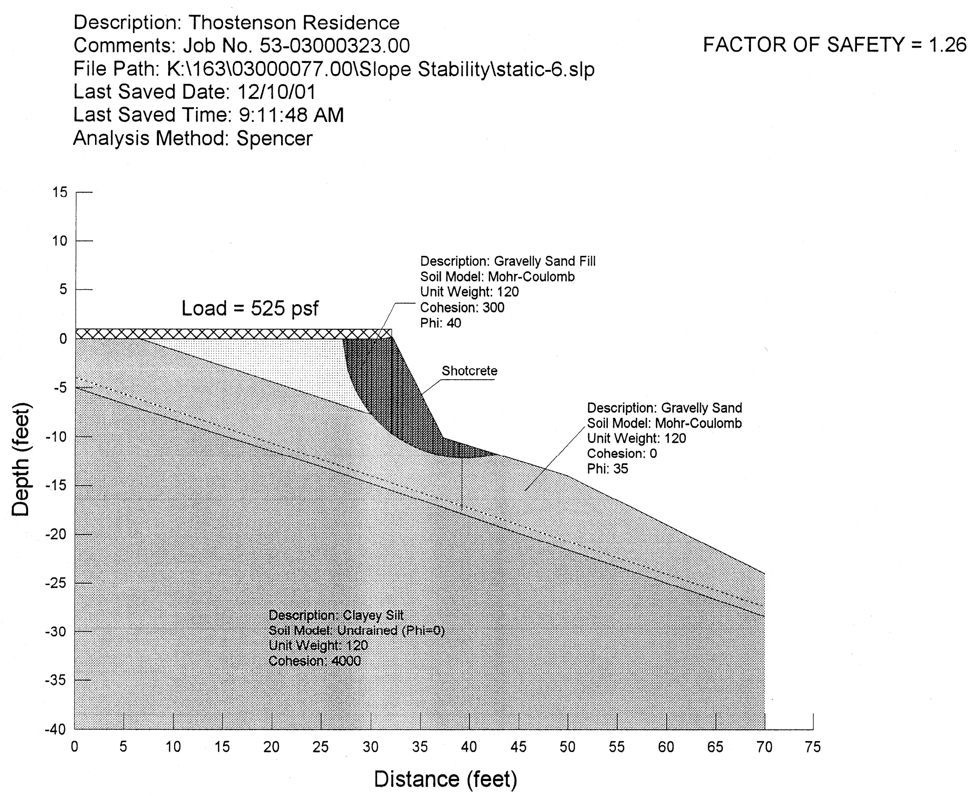

Figure 8: Static slope stability analysis, Section A

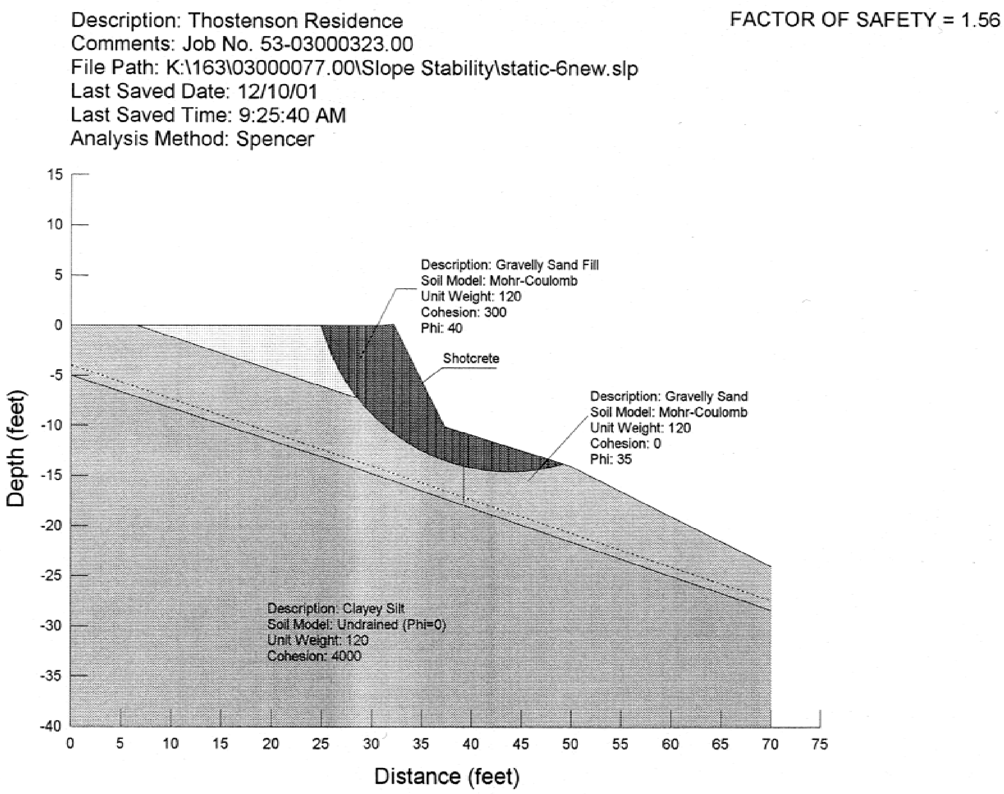

Figure 9: Static slope stability analysis - no surcharge, Section A

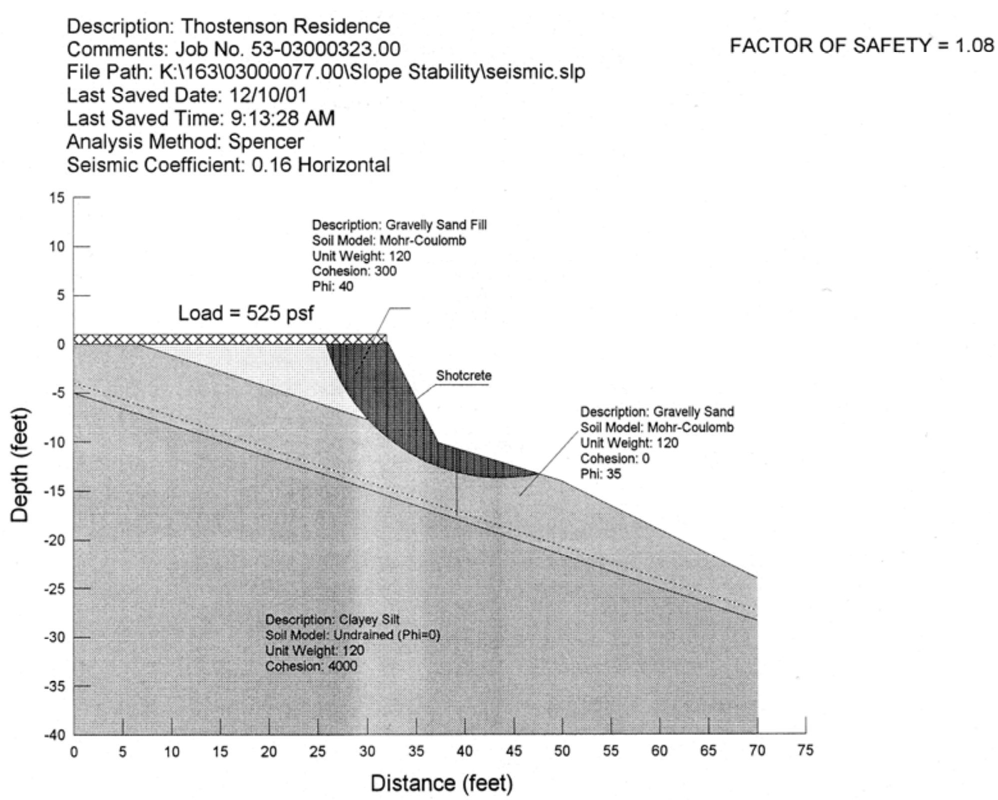

Figure 10: Seismic slope stability analysis, Section A

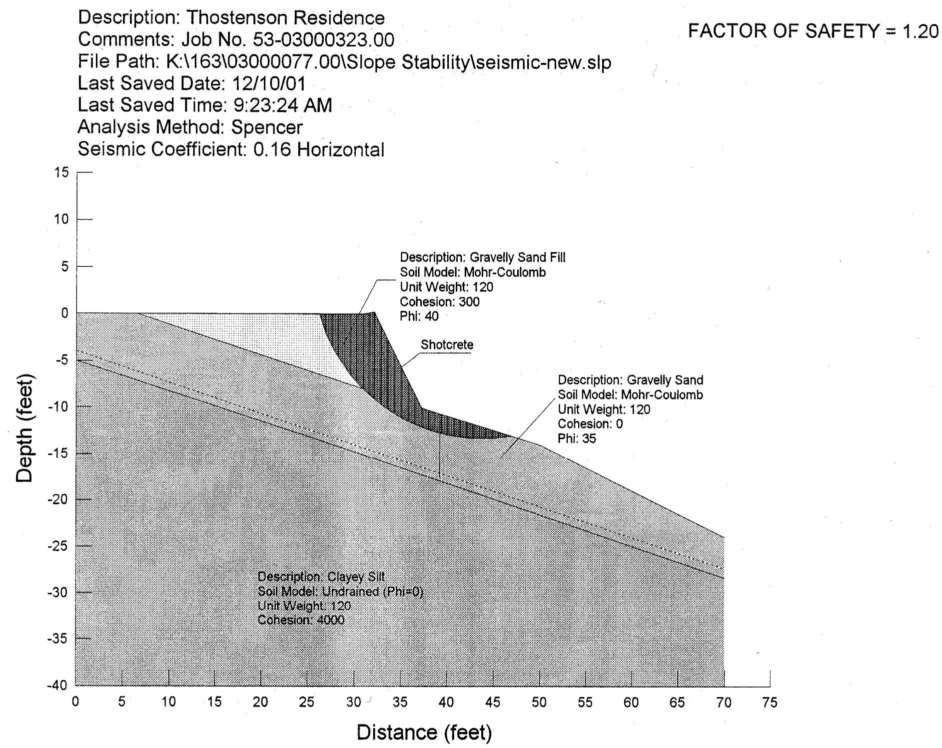

Figure 11: Seismic slope stability analysis -no surcharge, Section A

At Profile B the analysis indicates that an adequate seismic factor of safety of 1.22 is obtained if the full 17 kips of load from the column is applied to the slope (Figure 12).

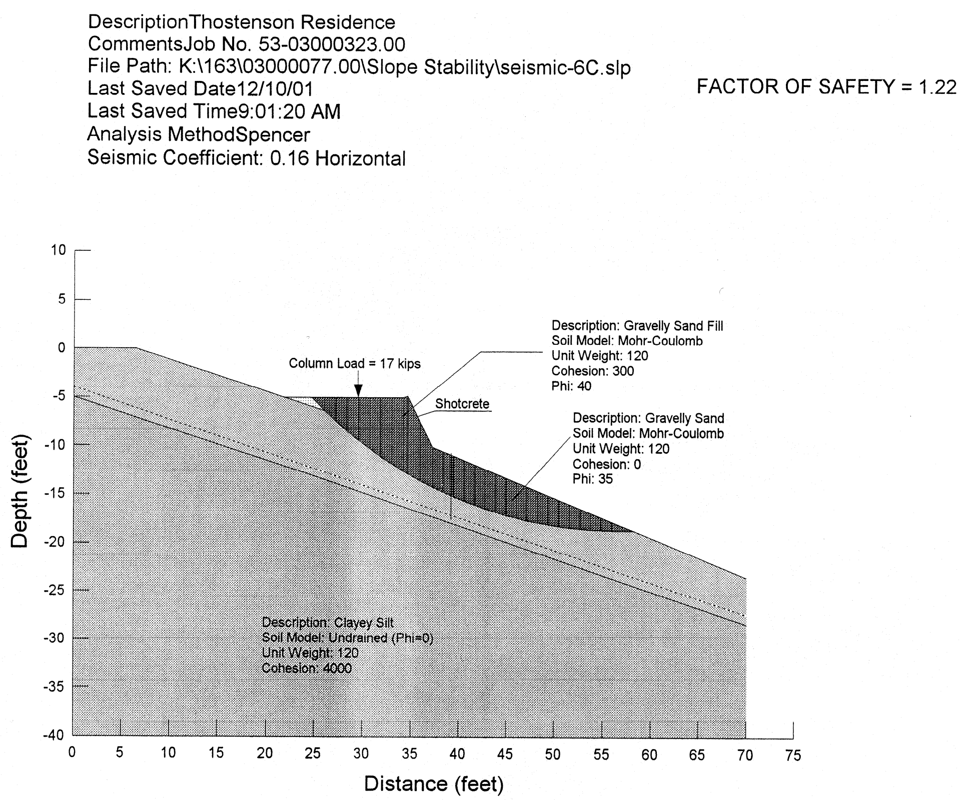

Figure 12: Seismic slope stability analysis, Section B

It should be noted that the outer 1 to 2 feet of soil on any portion of the slope, and particularly on the steep portion just below the pool, is vulnerable to weathering, sloughing and raveling over long periods of time due to deterioration from freeze-thaw and wet-dry cycles. This phenomenon cannot be adequately modeled using SLOPE/W software. The tendency for this shallow sloughing can be minimized if portions of the slope not already covered with shotcrete are protected with additional shotcrete coverage, or are protected by planting shrubs that will help control the erosion and sloughing processes.

CONCLUSIONS AND RECOMMENDATIONS BASED ON GEOTECHNICAL INVESTIGATION

- Although the position and depth of embedment of all the piers supporting the swimming pool are not known with any certainty, the available evidence from historical reports and from recent observations suggests that the number and lengths of the piers may be sufficient to transfer the pool load through the fill into the more competent underlying native gravelly sand layer. Slope stability analyses for this "No Surcharge" scenario have produced adequate factors of safety for the static and seismic loading cases. Accordingly, no mitigation is recommended for pool support.
- Long term wetting/drying and freeze-thaw cycles may cause very shallow sloughing failures on the steep slope immediately below the pool. The potential for this activity can be minimized by applying a coat of shotcrete to the slope face at locations where shotcrete does not already exist. Alternatively, planting vegetation on the exposed surfaces will aid in reducing the potential for future erosion and sloughing. The existing English Ivy is considered a nuisance species, but can serve as a protective vegetative cover. Other species can be used, as recommended in the Washington Department of Ecology Publication 93-30 "Slope Stabilization and Erosion Control Using Vegetation" (1993). Excavated pockets in the face of the slope around the piers can be backfilled using Controlled Density Fill, which is a very weak concrete mix that is also called "flowable fill". When cured, it has the consistency of a very stiff cohesive soil, and can be excavated later if needed. Forms will be needed to keep it in place when first poured.
- At the 18-inch column location, downward movement of the column in the past was likely the result of downhill creep of the upper loose fill that appears to be slightly steeper just below the column than elsewhere in the vicinity. The fill at the column location appears to be less well compacted than the fill beneath the pool. While the base of the column seems to be embedded at least 6 feet, the loose and disturbed fill apparently extends deeper at this location, and the upper few feet of the underlying native gravelly sand may have also have been disturbed by erosion activity in the past. At this location, mitigation using pin pile installation as underpinning for the existing column. The pin piles should be installed around the perimeter of the existing column, and transfer the load from the column to the piles. The pin piles should consist of galvanized (or otherwise corrosion-protected) 2-inch diameter schedule 40 steel pipe driven at least 5 feet into the very stiff to hard gray silty clay. In addition to the minimum embedment requirement, the piles should be driven to a

penetration rate of at least 1 inch per minute for 3 consecutive minutes using a 90-pound air hammer. The allowable load for such piles may be taken as 5 kips. Other types of limited access piles are available such as the Chance Anchor- type pile or the IBO (Injection Boring) rod or micropile. The IBO rod consists of hollow high strength steel that is drilled into the ground, then grouted in place through the hollow center. An allowable load of at least 10 kips is expected from a 2-helix Chance Anchor pile (one 8-inch diameter helix and one 10-inch diameter helix) or an IBO rod pile, provided that each is installed at least 5 feet into the very stiff to hard gray silty clay. Connections from the piles to the column must be designed to adequately transfer and evenly distribute the loads.

NEW COLUMN CONSTRUCTION

After several site visits and completion of the geotecnical investigation conducted by URS Corporation, and based on conclusions and recommendations from the geotechnical report, it was decided to keep the existing column in place and physically connecting it to the slab it used to support, and in addition building a new column near the existing column. For this purpose, North Coast Drilling Co. was selected to construct this new column and engaging the existing column to the slab.

For the construction of the new column, two A.B. Chance SS5 helical anchors hot dip galvanized (per ASTM A153) 1-1/2" square bars with single 10" diameter helix were utilized to be driven 20 to 25 feet into the existing ground. Both piles experienced an average torque for the last 3 feet of installation at 3,000 ft-lbs. The final torque to refusal was 4,000 ft-lbs. 4,000 ft.-lbs of torque correlates to approximately 40 kips of capacity.

SS5 Helical Anchors

Allowable torque capacity-5,500 ft-lbs.

Ultimate capacity for axially loaded foundation-55kips.

Note:

1. Hot dip galvanized per ASTM A153.
2. Lead and extension section and pilot point lengths are nominal; expect some variance.
3. Shaft material per ASTM A29 (latest revision) or mechanical equivalent.
4. Helix material low carbon steel meeting the general requirements of AISI, or ASTM A572 or A935, or ASTM a656 or A936.
5. Coupling bolts: 3/4" diameter x 3" long hex head per ASTM A320 grade l7.
6. Nominal spacing between helical plates is three times the diameter of the lower helix; expect some variation.
7. Manufacturer to have in effect industry recognized written quality control for all materials and manufacturing processes.
8. All welding to be done by welders certified under section 5 of the aws code D1.1.
9. See ICBO evaluation service inc., Evaluation report no. ER-5110 for allowable values and/or conditions of use concerning material presented in this document.

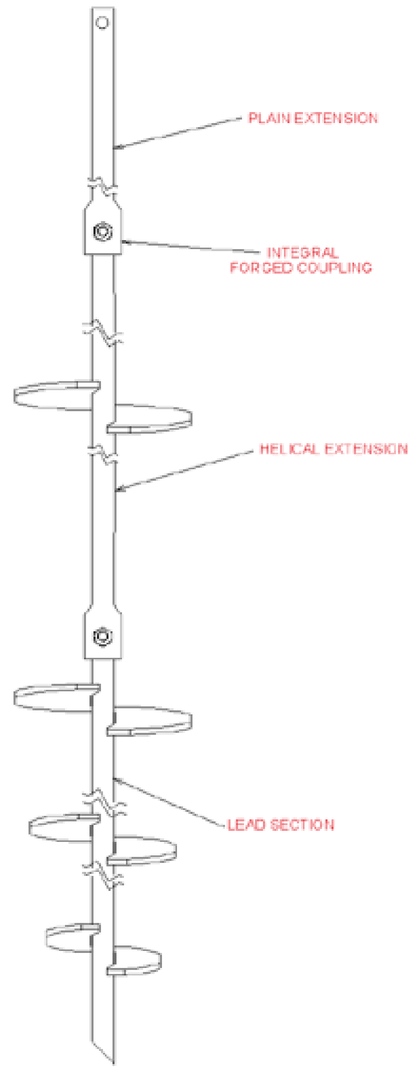

Figure 13: Typical Anchor Assembly

Once these piles were driven to a depth of 20 to 25 feet, they had tensile strength capacity of 70 kips (single 40 kips).

The support piles were then connected to a 4"×8"×3/4" steel plate. The column itself is a 6×6×15 square structural tubing with a length of 10 ft. A base plate with the size of 12"×12"×3/8" is welded to the bottom of the column. The column base plate is then bolted to the to the plate on top of the piles using four 7/8"×10" galvanized bolts. At the top of the column, a 3/8" steel plate 12 " long was welded to the top of the column and two 12"×6"×3/8" steel plates were welded on both sides of the top plate, effectively creating a channel-shaped structure on top of the steel tube column. This channel then embraced the existing beam and four 3/4"×4 1/2" wedge anchors (2 on each side) were used to secure the new column to the beam.

A compressive load was placed on each pile by turning these nuts alternatively with a 16" long wrench to refusal. The load was transferred directly from the piles to the concrete beam prior to placing concrete pile cap.

Finally, the entire bottom system is boxed in a 24"×24"×12" reinforced concrete pile cap. For reinforcement #4 bars were used 4" O.C. both ways. Figure 14 shows the details of concrete pile cap.

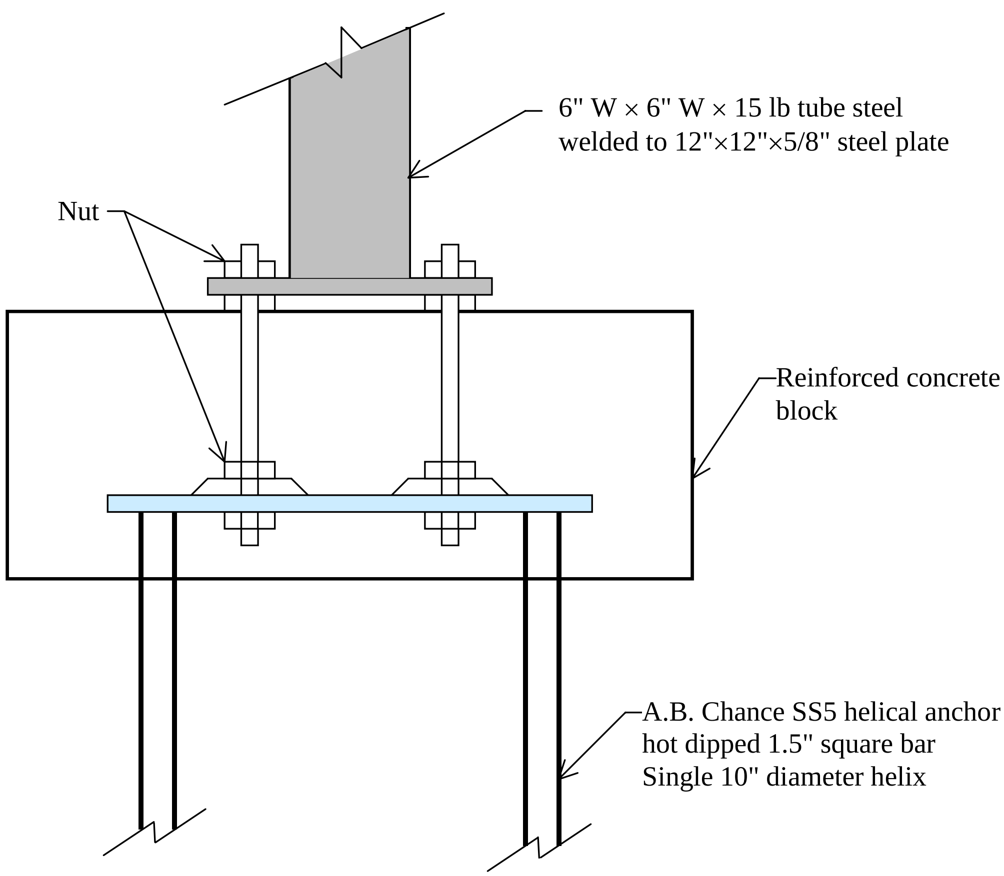

Figure 14: Details of concrete pile cap

Figure 15 shows the sketch of the new column and some of its details

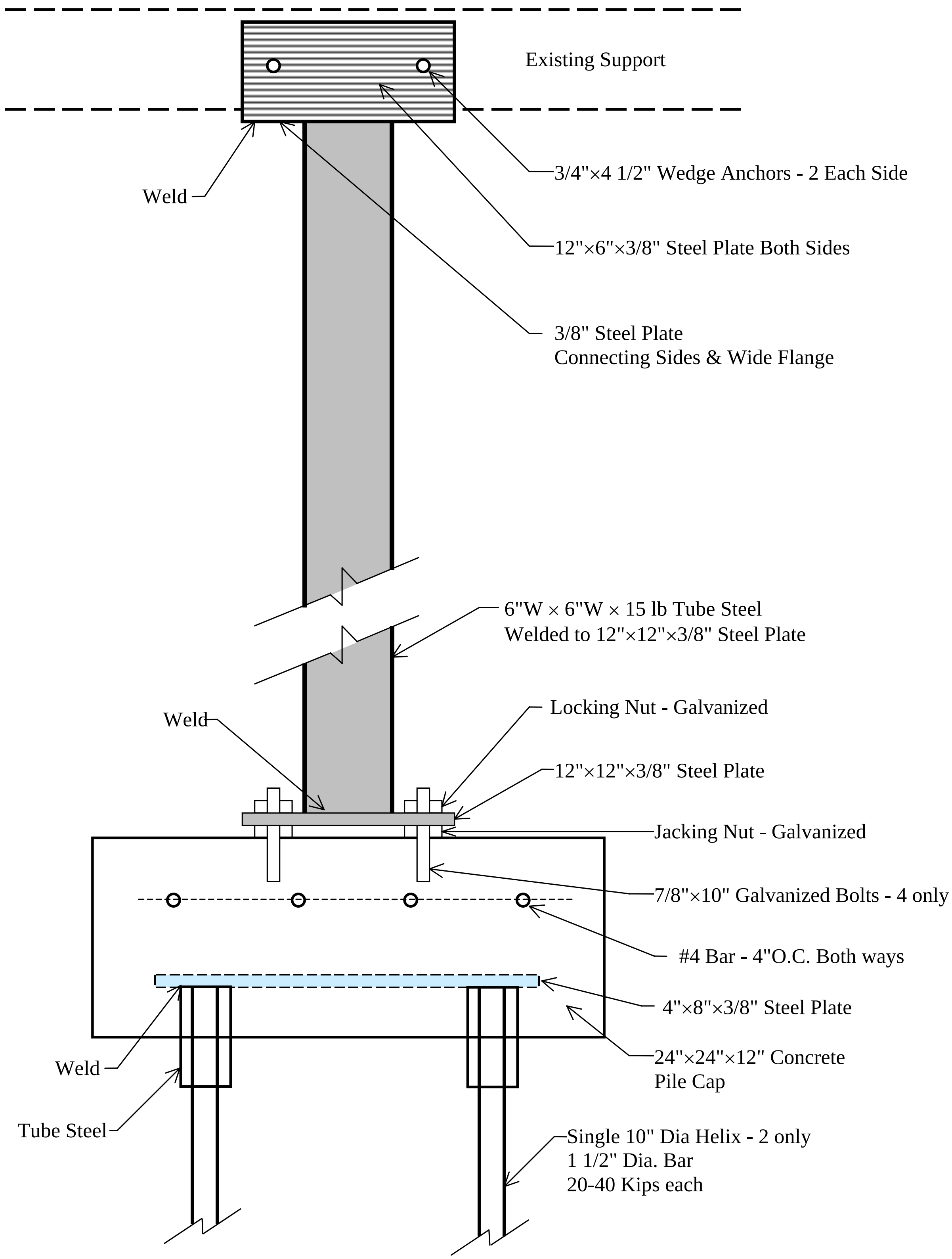

Figure 15: Sketch of the new column and details

Further details on top of the concrete pile cap, is a 6"W×18"∠×3/8" flat bar was welded to the top of driven piles, and two 4"W×12"∠×1/4" steel channel was connected to the top of the flat bar.

From AISC (American Institute of Steel Construction) Manual of Steel Construction, Table C-C2.1, the recommended design value for the *K* factor for a column with both ends fixed is 0.65.

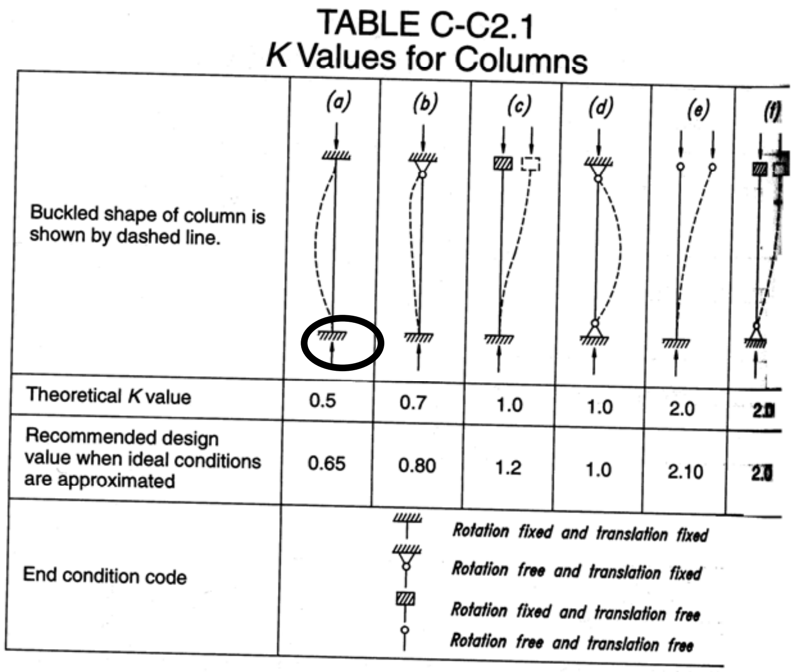

TABLE C-C2.1
K Values for Columns

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line.						
Theoretical *K* value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.10	2.0
End condition code	Rotation fixed and translation fixed Rotation free and translation fixed Rotation fixed and translation free Rotation free and translation free					

Figure 16: *K* values for columns (from AISC)

Then:

$$KL = (0.65)(10\text{ ft.}) = 6.5\text{ ft.}$$

Referring to the AISC table for square structural tubing, the design axial strength can be determined. For a 6×6×15 tube, with a *KL* of 6.5, the axial strength is approximately 155 Kips (155,000 lbs), which is sufficient to carry the load imposed from the slab above is determined.

Figure 17 shows the table from AISC used to determine the design axial strength of the column.

F_y = 46 ksi

COLUMNS
Square structural tubing
Design axial strength in kips (ϕ = 0.85)

Nominal Size		7×7						6×6					
Thickness		5/8	1/2	3/8	5/16	1/4	3/16	5/8	1/2	3/8	5/16	1/4	3/16
Wt./ft		50.81	42.05	32.58	27.59	22.42	17.08	42.30	35.24	**27.48**	**23.34**	**19.02**	**14.53**
F_y		46 ksi											
Effective length KL (ft)	0	584	485	375	317	258	196	486	407	316	268	219	167
	6	553	461	357	302	246	188	451	379	295	251	205	157
	7	543	452	351	297	242	185	438	369	288	245	200	153
	8	531	443	344	291	237	181	425	358	280	239	195	149
	9	518	432	336	285	232	177	410	346	271	231	189	145
	10	503	421	327	278	226	173	394	333	262	223	183	140
	11	488	409	318	270	220	168	377	320	252	215	176	135
	12	472	396	308	262	214	164	359	306	241	206	169	130
	13	454	382	298	254	207	159	341	291	230	197	162	124
	14	437	368	288	245	200	153	322	276	219	187	154	119
	15	418	353	277	236	193	148	303	260	207	178	146	113
	16	399	338	265	226	185	142	284	245	195	168	138	107
	17	380	322	254	217	177	136	265	229	184	158	131	101
	18	361	307	242	207	170	130	246	214	172	148	123	95
	19	342	291	230	197	162	124	227	199	160	138	115	89
	20	323	276	218	187	154	118	210	184	149	129	107	83
	22	285	245	195	167	138	107	175	155	127	111	92	72
	24	248	215	172	148	123	95	147	131	107	93	78	61
	26	214	187	151	130	108	84	125	111	91	80	67	52
	28	184	161	130	113	94	73	108	96	79	69	57	45
	30	161	140	113	98	81	63	94	84	69	60	50	39
	32	141	123	100	86	72	56	83	73	60	53	44	34
	34	125	109	88	76	63	49	73	65	53	47	39	30
	35	118	103	83	72	60	47	69	61	50	44	37	29
	36	111	97	79	68	57	44		58	48	41	35	27
	37	106	92	74	64	54	42			45	39	33	26
	38	100	87	71	61	51	40				37	31	24
	39	95	83	67	58	48	38						23
	40	90	79	64	55	46	36						
Properties													
A (in²)		14.9	12.4	9.58	8.11	6.59	5.02	12.4	10.4	8.08	6.86	5.59	4.27
I (in.⁴)		97.5	84.6	68.7	59.5	49.4	38.5	57.3	50.5	41.6	36.3	30.3	23.8
r (in.)		2.56	2.62	2.68	2.71	2.74	2.77	2.15	2.21	2.27	2.30	2.33	2.36

Note: Heavy line indicates Kl/r of 200.

Figure 17: AISC table for design axial strength of square structural tubing

Figure 18 shows the top of the new column connected to the existing beam as well as a new bracket physically connecting the existing concrete column to the existing concrete beam.

Figure 18: The existing and new columns connected to the beam

Figure 19 shows the existing concrete column next to the newly constructed column.

Figure 19: The existing and new columns

Figure 20 shows the new column and the gap at the top of the existing concrete being filled to make physical connection to the slab above.

Figure 20: The new column and repair of the existing column

The new steel column was primed and painted to resist corrosion.

If there are any questions, please feel free to contact me.

Respectfully submitted,

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