How to Design Piles Against Uplift

By Raymond Lundgren

IT IS OFTEN necessary to design structures to resist large lateral loads. These lateral loads may result from cantilevered construction, earth pressures, wind, and—of great importance in certain areas — earthquake loading. If the structure is pile-supported and is subjected to overturning forces, it may be necessary to design the piles to resist uplift.

The uplift capacity of piles is generally controlled by the unit shearing resistance, either adhesion or friction, available at the interface of pile and soil. The adhesion between the pile and clay soils may be determined on the basis of the soil cohesion (one-half the unconfined compression strength) times a coefficient that is based both on experience and published data.[1]

For soft to stiff clays, the adhesion which can be developed is frequently as large as the soil cohesion. As the soils become stiffer and/or more brittle, however, the ratio of adhesion to cohesion becomes less than 1.0. For very stiff or hard clays, the adhesion may be considerably less than the cohesion. The strength-time and strain-time properties of the soil may also limit the adhesion. Soft clays have a tendency to creep under sustained, long-term loads. If the pile extends through both soft and stiff clays, the creep tendencies of the soft soils may result in a transfer of loads to the stiffer soils. The use of the reduced adhesion in soft

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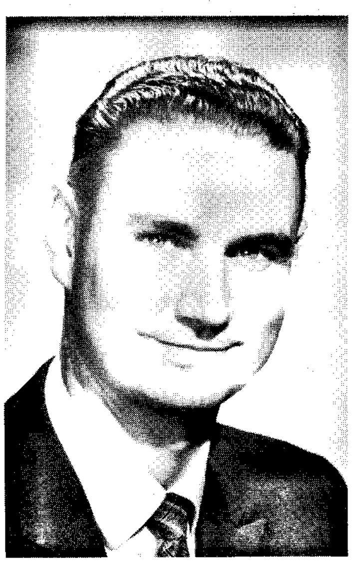

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clays is, therefore, advisable when the pile will be subjected to long-term, uplift forces.

For piles in granular soils, the unit shearing resistance is proportional to the lateral earth pressure on the pile and is estimated from the following formula:

$$S = K_H \times \overline{p}_{avg} \times \tan\delta \qquad \textbf{Equation (1)}$$

where S = unit shearing resistance, pounds per square foot

K_H = coefficient of lateral earth pressure at the interface of the pile and the soil

$\overline{p}_{avg}$ = average effective overburden pressure (psf) at the midheight of the section of pile being considered

$\tan\delta$ = coefficient of friction between pile and soil

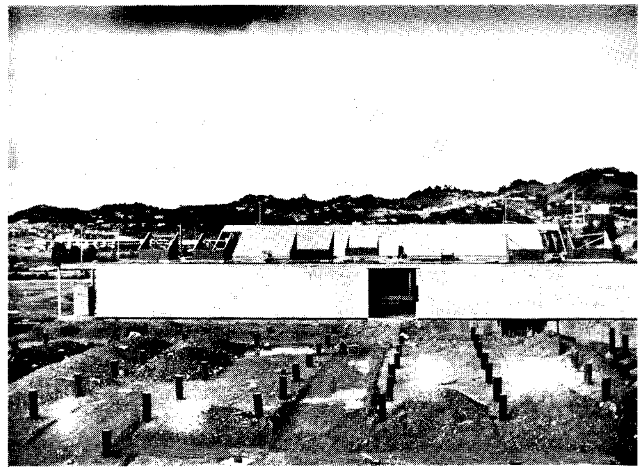
Equation (1) can also be applied to nonplastic silts or sandy silts.

Two specific projects where pressure-treated wood piles were utilized for foundation and uplift support are presented. The first case illustrates our design approach where clay-type soils are prevalent, and the second case where granular soils are prevalent.

Case History I

Pressure-treated Class B timber piles were used to support the Marin Co-op Shopping Center in Corte Madera, California. Piles were used to eliminate differential settlement as well as to provide short-term uplift resistance during seismic loading. The long-term uplift resistance of piles beneath a proposed retaining wall was also required for this project. The piles were end-bearing piles driven into weathered sandstone. The sandstone was overlain successively by stiff silty and sandy clays, soft silty clays (locally termed Bay Mud), and loose rocky fill. Typical soil profiles at the site are shown in Figure 1.

Because spudding through the fill was anticipated, any small uplift resistance that might have been provided by the fill was neglected. The design uplift resistance is provided in both the soft Bay Mud and the underlying layer of stiff clay. For short-term seismic loading, the adhesion was assumed equal to the cohesion for both the soft and the stiff clays. During long-term loads, however, the soft Bay Mud would tend to creep, as previously discussed. A reduced value of ad-



Pressure-treated timber piles at the Marin Co-op Shopping Center, Corte Madera, California.

LONG - TERM UPLIFT CAPACITY FOR PILES BENEATH RETAINING WALL

Typical Soil Conditions - Retaining Wall Location (12" pile; 27' total)	Unit Shearing Resistance (psf)	Uplift Capacity (kips)
4' Loose, Clayey Sand, Sandy Clay and Rock Fill	- 0 -	- 0 -
15' Soft, Silty Clay (Bay Mud) c = 250 psf	125	5.2
8' Stiff, Silty and Sandy Clay c = 1500 psf	1500	29.6
9" Weathered Sandstone		Σ = 34.8 kips

SHORT - TERM UPLIFT CAPACITY FOR PILES BENEATH BUILDING

Typical Soil Conditions - Building Location (12" pile; 14' total)	Unit Shearing Resistance (psf)	Uplift Capacity (kips)
4' Loose, Clayey Sand and Rock Fill	- 0 -	- 0 -
6' Soft, Silty Clay (Bay Mud) c = 250 psf	250	4.1
4' Stiff, Silty Clay c = 1500 psf	1500	14.8
9" Weathered Sandstone		Σ = 18.9 kips

FIG. 1 - COMPUTED UPLIFT CAPACITY FOR TREATED TIMBER PILES

MARIN CO-OP SHOPPING CENTER

CORTE MADERA, CALIFORNIA

hesion equal to 50 percent of the cohesion was, therefore, used. The computed uplift capacity for each case is shown in Figure 1.

Case History II

Pressure-treated Class B timber piles were used to support the new Coca-Cola Building in San Francisco, California. A typical soil profile showing dense sands underlying loose sands is shown in Figure 2. At this

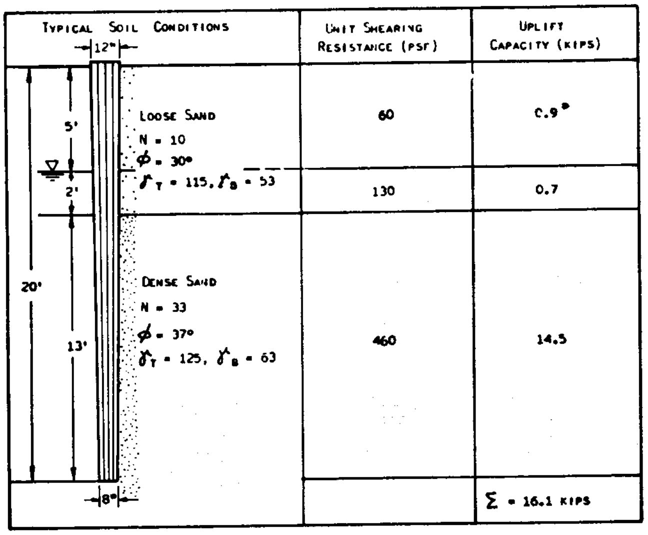


Fig. 2 - Computed Uplift Capacity for Treated Timber Piles
Coca Cola Building
San Francisco, California

site, piles were utilized to resist short-term uplift loads due to wind or seismic loading. The unit shearing resistance was estimated using equation (1).

For the uplift computations shown in Figure 2, values of K_H equal to 0.5 and 0.8 were used for the loose and dense sand layers, respectively. Higher K_H values could have been used, as discussed subsequently, if predrilling had not been used. The angle of friction between the soil and the wood, δ, was assumed to be three-fourth of the internal angle of friction, ϕ.

In general, the uplift capacity of a pile embedded in granular soils is difficult to estimate. Primarily, this difficulty is encountered because the lateral earth-pressure coefficient, K_H, may increase significantly due to the high lateral stresses induced in the soil by driving a displacement-type pile. Ireland (1957)[2] computed K_H values for uplift tests made on driven step-taper piles with steel shells and found the lateral earth-pressure coefficient to vary from 1.79 to 3.70. The soils were loose to medium dense sands with an average blow count, N, of 10 with $\phi = 30°$. Ireland believed that a value of 1.75 as a lateral earth-pressure coefficient would be conservative, but should be verified by field tests.

Piles for the Coca-Cola project were predrilled to within two feet of tip elevation. The lateral stresses induced by driving were thus minimized, and the conservative values for K_H mentioned previously were necessary.

In lieu of using equation (1), the following empirical relationship[3] suggested by Meyerhoff may be used to estimate the unit shearing resistance in granular soils:

$$S = \frac{N}{50} \qquad \textbf{Equation (2)}$$

where S = unit shearing resistance (tons per square foot)

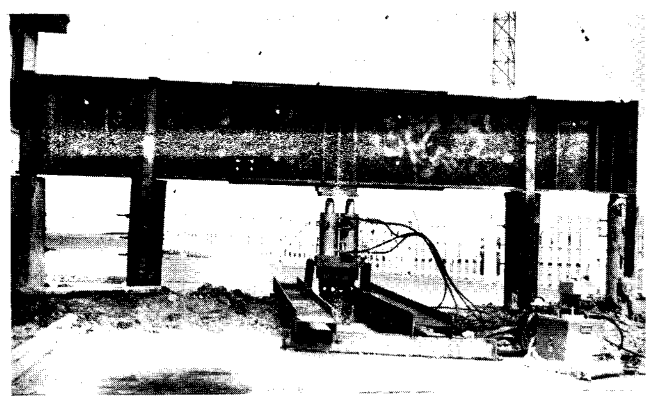
N = average standard sampler penetration along the section of the pile being considered.

Meyerhoff recommended an upper limiting value of S equal to one ton per square foot. Equation (2) was developed on the basis of observations for driven-displacement piles. Hence, the formula would not be directly applicable where predrilling, spudding, or jetting are used as aids in driving piles. In addition, equation (2) was developed for piles driven in saturated soils. The formula is, therefore, conservative in cases where the free groundwater level is at some depth below the top of the supporting soils. In general, the use of equation (2) results in higher computed uplift capacities for short piles but lower capacities for long piles.

Other Methods

There are also a number of other methods for computing the uplift capacity of a single pile. Each method may yield different results and all methods are sensitive to the choice of the appropriate soil parameters. The approach that is generally used is to compute capacities using more than one method and to compare the results with experience. For large jobs, where uplift is critical and where there are large potential savings in cost, the soil investigation should be supplemented by a full-scale, pull-out test.

In addition to providing uplift resistance for structures, timber piles are frequently used as reaction piles in performing standard pile loading tests. One such example (Case History III) is the U. S. Coast Guard Barracks and Mess Hall structure at the San Francisco International Airport. Pressure-treated Class B timber



Load test set-up at the U. S. Coast Guard Barracks and Mess Hall, San Francisco International Airport.

piles, designed for an allowable downward load of 50 tons, were used throughout this project. The load-test arrangement and the soil profile are shown on Figure 3 along with a photograph. The test piles were loaded in increments to an ultimate load of 180 tons. At this load, the reaction piles supported an average pull-out load of 45 tons per pile. During the loading process, the upward deflections of the reaction piles were observed and recorded. The average results for the four

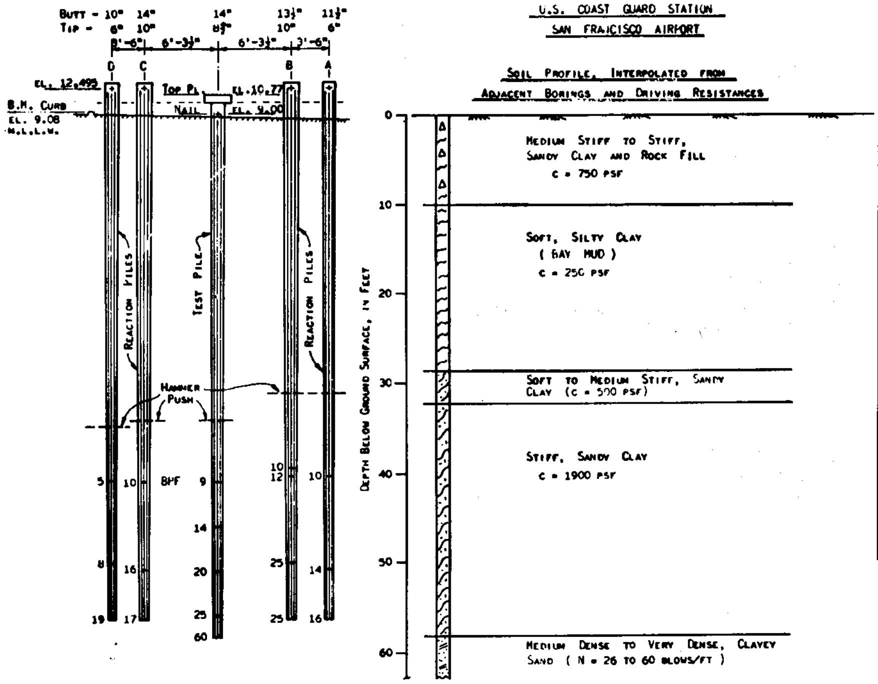


FIG. 3 - LOAD TEST SET-UP AND SOIL CONDITIONS
U.S. COAST GUARD STATION
SAN FRANCISCO AIRPORT

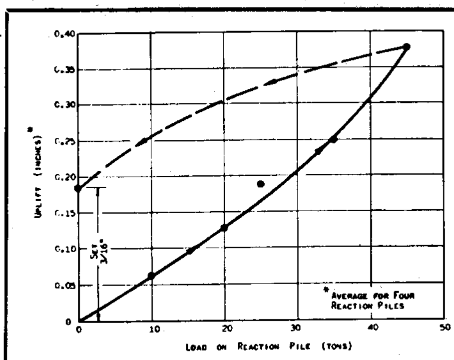


FIG. 4 - LOAD - UPLIFT CURVE FOR REACTION PILE
U.S. COAST GUARD STATION
SAN FRANCISCO, CALIFORNIA

reaction piles are plotted as uplift vs. load in Figure 4. Although the reaction piles were not loaded to failure, the shape of the load curve indicates that failure might have occurred at a load of from 50 to 60 tons per pile.

Uplift resistance for the reaction piles was computed on the basis of the soil properties. Adhesion values in the upper 32 feet were estimated to equal the soil cohesion. For the stiff sandy clays below this depth, the adhesion would be less than the soil cohesion. A value of adhesion equal to 1,500 psf was, therefore, used. The resulting computed ultimate capacity was 60 tons per pile, which is in good agreement with the load-test data.

Three additional examples of loads applied to timber reaction piles are given in Table 1. In none of these cases did the reaction piles deflect an appreciable amount. Hence, the maximum load sustained by each reaction pile (column 6) is less than the ultimate pull-out load which could have been developed. In columns (7) and (8) of Table 1, the theoretical ultimate

Table 1. Data from Pile Load Tests Using Wood Reaction Piles

Project -1-	Soil Conditions: Depth below ground surface (ft.) -2-	Soil Conditions: Soils -3-	Reaction pile diameter (in.) -4- Tip	Reaction pile diameter (in.) -4- Butt	Reaction pile embedment length (ft.) -5-	Maximum load applied to reaction pile (tons)* -6-	Theoretical ultimate load (tons) A** -7-	Theoretical ultimate load (tons) B*** -8-
I. Port of Sacramento, Sacramento, California	0 to 18	Medium stiff clay, C = 500 psf	9	15	64	100	80	130
	18 to 39	Medium dense silt and fine sand, N = 16						
	39 to 45	Medium dense silt, N = 26						
	45 to 58	Dense silt, N = 38						
	58 to 64	Medium dense fine sand, N = 24 NOTE: Groundwater level at ground surface.						
II. Bank of California, Sacramento, California	0 to 16	Medium stiff silty clay to loose, clayey silt C = 625 psf	6	14	58	45	55	55
	16 to 58	Medium stiff silty clay to medium dense clayey silt, C = 785 psf NOTE: Groundwater level at 20 ft.						
III. 19th Avenue Shopping Center, San Mateo, California	0 to 4	Medium dense silty sand, N = 30	10.5	12	40	27.5	47.5	47
	4 to 20	Soft Bay Mud, C = 250 psf						
	20 to 24	Medium stiff silty clay, C = 750 psf						
	24 to 29	Dense clayey sand, N = 33						
	29 to 33	Very stiff sandy clay, C > 2000 psf						
	33 to 40	Dense clayey sand, N = 33 NOTE: Groundwater level at 3 ft.						

*No appreciable deflection of the reaction piles had occurred at the maximum load.

**Computed using adhesion, C_A in clays and $S = \frac{N}{50}$ for granular soils.

***Computed using adhesion, C_A in clays and $S = K_H \times \bar{P}_{(avg)} \times \tan \delta$ in sands and silts.

($\delta = 3/4\ \phi$ and $K_H = 1.75$)

"... capacity was 60 tons per pile, which is in agreement with the load-test data."

load is computed using adhesion values for clayey soils and equations (1) or (2) for granular sands and silts. It is interesting to note that, for Case I, equation (2) yields an unrealistically low value for uplift capacity; for Case III, equations (1) and (2) yield values which compare closely.

Another example of the resistance of timber piles to uplift concerned the attempt of LeBoeuf Dougherty Contracting Company (piling contractors) to remove an old timber pile at Mare Island, California. The pile was 90 feet in length and extended through 65 feet of soft Bay Mud into 25 feet of medium stiff to stiff clay. The pile finally pulled out after application of a 150-ton load for two hours, and in coming out it pulled up a cylinder of soil 30 inches in diameter around the pile. In this case, the adhesion which had developed at the soil-pile interface exceeded the soil cohesion a short distance away. The pile did not fail structurally during pull-out.

Conclusions

In conclusion, it is possible to safely design timber piles against uplift. The ultimate uplift capacity may be computed by summing the soil-pile unit shear strength of each layer times the contact area. For short-term, uplift forces, all soils penetrated may be assumed to provide resistance. A reduction factor should be applied to the strength of stiff to hard clays as well as to those soils disturbed by spudding, jetting, or pre-drilling. For long-term uplift forces, the tendency for creep or relaxation in softer soils must also be considered. The unit shear strength in sands may be computed by equations (1) or (2). Equation (1) should be used if the soil properties are well known; but if only sampler penetration resistance is known, equation (2) may be used provided that an adequate safety factor is applied. The examples in Table 1 indicate that values of $\delta = \frac{3}{4}\phi$ and $K_H = 1.75$ or greater may be appropriate for use in equation (1).

It should be noted that this article deals only with ultimate load and, in all cases, an appropriate factor of safety must be applied. Furthermore, the uplift capacity of the pile group must also be checked and the allowable load reduced where the group efficiency is less than 100 percent. It should further be noted that this discussion applies only to the resistance developed between the soil and the pile; both the pile and the connection to the superstructure must be structurally designed to resist the imposed loads. ■

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