Structural Behavior of Driven Piling

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A review of the structural behavior of driven piles is made, and it is shown that, except for piles that fail because of improper construction or piles that deteriorate in service, there are very few reports of structural failure. The various reasons for this excellent record are examined, and the conclusion is that one of the principal reasons is the remarkable loadcarrying capacity of damaged piling. Several case histories are described involving the behavior of damaged piling under load and load tests on pile groups. The implications of these findings are discussed in relation to the allowable stresses used in practice, and a few situations are cited where it appears that the allowable loading on driven piles could be safely increased.

•VERY FEW FAILURES of pile foundations can be attributed to structural failure of the piling. Except for piles that fail because of improper construction or piles that deteriorate in service, the few reports that have been published involve piles that failed because the actual pile loading greatly exceeded the design loading. This paper examines the reasons for this excellent record. It investigates the various types of damage that occur during pile installation and the effects of damage on pile capacity. Finally, the allowable stresses used in practice are briefly surveyed. The discussion is limited to driven piles.

There are a few reports of structural pile failure due to dragdown loading resulting from settlement of the surrounding soils. One case involves 85-ft-long timber piles driven through 40 ft of fill and 28 ft of soft bay mud to end-bearing on decomposed rock. The tops of the piles settled several inches and, because the tips were restrained, it is presumed that the piles broke. The potential dragdown load was estimated to be almost 200 tons per pile (1). Chellis (2) reports a similar case involving 100-ft-long steel H-piles driven end-bearing to rock through a deep bed of plastic clay. The area was brought to grade with 15 ft of slag fill, which also supported a floor slab carrying very heavy loads of armor plate. Within a year settlements of 1 ft occurred, and the H-piles jackknifed. The total loads in this case were estimated to be 350 tons per pile.

Dragdown loads of this magnitude are quite possible. In Norway, Bjerrum and his colleagues $(\underline{3})$ have measured dragdown loads of 120 metric tons on a 12-in. steel pipe and 300 metric tons on a 20-in. pipe. The piles were driven through 100 ft of fill and soft clay to bearing on rock.

Another type of structural pile failure is reported to be the cause for large settlements of an ore-storage dock ($\underline{4}$). Step-taper piles, used to support a rigid concrete slab, were driven to end-bearing through 80 ft of fill and soft clay. Some of the piles stopped in an underlying layer of hardpan while neighboring piles were able to penetrate through the hardpan and reach bedrock. This condition resulted in piles of unequal stiffness. When the slab was loaded a gradual, progressive failure took place, presumably because the stiffer piles became overloaded and failed. The piles had an 8-ftlong 10.75-in. O. D. pipe section at the tip and were designed for a 150-ton capacity. This is a very heavy loading for this pile in an end-bearing situation and, no doubt, this was a contributing factor to the failure. Failure probably occurred in shear immediately above the composite connection. Settlements of almost 2 ft were observed.

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Perhaps a more extensive study would uncover other reports of failures and, of course, some failures go unreported; but judging by the scarcity of published reports, there are remarkably few structural failures of driven piles.

PILE BEHAVIOR

The first serious loading a pile is subjected to occurs when it is driven into the ground. In most cases the stresses that a pile experiences during driving exceed the design stresses and are the highest stresses that the pile ever experiences. If the pile can withstand these forces without damage, it has, in effect, been pretested. A few measurements of driving stresses in piles are reported in the following.

A second reason for the excellent record of the structural behavior of driven piling is that piles seldom receive their full design loading. The principal reason for this is that the usual design provisions for live loading are quite conservative and are seldom, if ever, realized in practice. However, in addition to this there is also a small but growing body of evidence to indicate that the actual column loads may never reach the piles. It seems that when piles are driven in groups and capped a significant proportion of the column load is transferred directly from the pile cap into the soil.

Evidence of this phenomenon is demonstrated in tests by Vesic $(\underline{5})$ on large-scale models of pile groups in sand. The model piles were 4-in. diameter aluminum tubes that were jacked into the sand to a depth of 60 in. A concrete cap was then cast directly on the sand surface. The test loads were regulated with an electronic proving



Figure 1. Load-displacement during Test p-42 (5).

ring, and the pile loads were measured with strain gages. Test results of a 4-pile group in medium-dense sand are shown in Figure 1. The proportion of the total load carried by the cap increased from about 19 percent at a displacement of 0.1 in. to about 23 percent at a displacement of 1.0 in. This test is fairly typical.

Very few field tests have been performed on pile groups. One of the few published reports ($\underline{6}$) concerns a 9-pile group of 12-in. pipe piles spaced at 3-ft centers. The piles were pushed 19 ft into a deep deposit of soft organic silt. A shallow excavation was made in order to place the cap directly on the organic silt layer. Test loads were applied with a hydraulic jack, and the soil reactions at the bottom of the pile cap were measured with 3 earth pressure cells. As shown in Figure 2, at loads less than 140 metric tons about 10 percent of the group load was carried by the base reaction on the cap. At this point the cap had settled about 0.6 in. As the load was increased further, yield occurred and the





base reaction increased rapidly. At failure, slightly more than 20 percent of the total load was carried by the bearing pressure at the base of the pile cap.

These data show that, even for very soft soils, the load reaching the piles is less than the full applied load.

Another favorable circumstance that contributes to the excellent record of the structural behavior of driven piling is that piles are frequently stronger than is recognized by our present design procedures. The concrete core in a concrete-filled pipe pile, for example, benefits from the confinement provided by the steel pipe. Recent experiments have shown that where concrete-filled steel tubes act as short columns (KL/r < 35), the ultimate load-carrying capacity is 20 to 30 percent greater than that predicted by theory (7). This may be of particular interest, because some building codes now permit piles to be designed on the basis of ultimate strength. Also, the materials used to construct piles are usually stronger than we credit them with being. Design stresses are based on specified minimum strengths, and it is obvious that the average strength must be greater than the specified minimum. It is not unusual, for example, to find that the average yield strength of steel pipe exceeds the specified minimum by as much as 10,000 psi.

PILE DAMAGE

All of these factors act to minimize the possibility of structural failure. However, driving piles into the ground is a brutal process that frequently causes damage to piling, and this increases the danger of structural failure. Bent piling and tip damage are the more prevalent and serious types of damage. Damage to the tips of piles usually occurs because of overdriving, but it may also occur if the pile hits an obstruction. For concrete piles casy driving can be as harmful as hard driving. When concrete piles are driven into a soft stratum, tension cracking sometimes occurs because a tensile stress wave is reflected up from the pile tip (8).

A bent pile usually results from hitting an obstruction, but it appears that bending can also be due to a variety of other causes. The driving of slender piles into soft soils has been studied at the Norwegian Geotechnic Institute (9), and some of the factors that may contribute to pile bending are shown in Figure 3. Bending will obviously increase the danger of buckling, and this can be a serious problem in soft clays because of the limited ability of the soil to provide lateral support.

Hanna $(\underline{10})$, based on a study of bending of long steel H-piles, suggests that bending results from a mechanism that is inherent in all pile-driving work (Fig. 4). As a pile punches through the overburden, an asymmetric failure pattern develops at the pile tip, and this results in an eccentric tip reaction. This causes a bending moment at the pile tip that may be sufficient to initiate bending even under light driving conditions in uniform soils. The pile will have a tendency to bend in the direction of newly driven neighboring piles around which the shear strength of the clay is previously reduced.

In trenches and shafts the displaced clay will primarily be squeezed up in the bottom of the excavation, and the piles will consequently

have a tendency of a curva-

ture against the centerline of the excavation.



Along the walls of an excavation, the displacement will take place mainly in the direction of the excavation; and the pile will tend to bend away from the wall.

Because of their weight, batter piles will tend to bend in a downward direction.

Figure 3. Causes of pile bending (9).

Another factor that may cause bending is the misalignment of spliced sections. When the leads of the driving rig are spliced in, it is particularly difficult to control alignment. Finally, there are certain pile joints and connectors that either permit limited rotation or have reduced moment restraint. Bending sometimes occurs at these points of inherent weakness.



Figure 4. Idealized mode of soil displacement during driving (10).

The behavior of damaged piling under loading is difficult to determine because surprisingly few test data are available. Load tests are hardly ever performed on piles that are known to be damaged and, unfortunately, piles that cannot be inspected are seldom extracted after load testing to investigate for possible damage. However, information that is available indicates that damaged piles have remarkable load-carrying capacity.

A search of the literature has uncovered only a few reports of load tests on piles with tip damage $(\underline{11}, \underline{12}, \underline{13})$. All of these involve steel H-piles that were overdriven into very dense material. The following example is typical of these test results.

Several steel H-piles, including one pile instrumented with strain gages, were driven through the Chicago blue clays and into a very dense hardpan (<u>11</u>). The driving records for 3 piles are shown in Figure 5 together with a typical soil profile. The piles drove easily to a depth of 60 ft, where the driving resistance began to increase steadily as the piles penetrated into the firmer clays. The hardpan occurs at a depth of 80 ft, and very hard driving was required to penetrate into this stratum. Pile 02 was driven to a resistance of 125 tons by the Engineering-News formula and penetrated less than 2 ft into the hardpan. The other 2 piles were purposely overdriven and penetrated deeper into the hardpan. A maximum driving stress of 17,000 psi is reported for the instrumented pile (Pile B4). The piles were driven with a single-acting hammer with a rated energy of 30,225 ft-lb and equipped with a helmet and hardwood block cushion.

The load test results are shown in Figures 6 and 7. There are 2 significant points. The first is that the piles that were overdriven experienced less settlement and had greater load-carrying capacity than did Pile 02. The second is that the strain-gage measurements show a high rate of load transfer in the hardpan stratum.

At the conclusion of the load tests, the 2 piles that had been overdriven were extracted. The condition of the instrumented pile is shown in Figure 8. The manner in which the flange was folded at the pile tip would indicate that the pile hit a boulder, although no boulders were encountered in either of the 2 borings at the site. There is also a bend in the flanges about 10 ft from the pile tip. As shown in Figure 9, the bottom of Pile B2 is completely mangled, and it is difficult to believe that this pile supported a 300-ton load. However, this pile was driven deeper than the other piles and, based



Figure 5. Pile driving records and typical soil profile for Piles B4, B2, and 02 (11).



Figure 6. Load-settlement for Piles B4, B2, and 02 (11).



Figure 7. Distribution of stress in Pile B4 (11).





Figure 9. Lower end of Pile B2 after extraction (11).

Figure 8. Lower end of Pile B4 after extraction (11).

on the strain-gage readings on Pile B4, it is likely that substantial loading did not reach the badly damaged portion of the pile.

Pile 02 was not extracted, so its condition cannot be definitely established. Because this pile was not overdriven, however, it seems likely that the pile was not damaged. As previously noted, this pile had the lowest load-carrying capacity.

On several occasions piles have been exposed during the driving of tunnels. Peck $(\underline{14})$, for example, reports encountering groups of timber piles in Chicago that were in such a twisted configuration that it was impossible to tell what piles belonged to a particular group. These piles had previously supported a 13-story apartment house without apparent signs of distress. As Peck points out, the significant point is that these piles are not different from those under hundreds of other buildings in Chicago.

A very interesting series of load tests performed on bent steel H-piles in Ontario, Canada, is reported by Hanna (10). Three H-piles were driven through a 140-ft deposit of glacial-lake clays. Except for the upper 35 ft, which were desiccated, the soils appear to be normally consolidated. As shown in Figure 10, the soils just below the desiccated layer are quite soft, with shear strengths on the order of 500 psf. Bedrock, a shale, occurs at a depth of 150 feet. Overlying the bedrock is a thin layer of dense sandy till.

The piles were driven to a final resistance of 40 blows per inch with a diesel hammer having a rated energy of 39,700 ft-lb. The driving resistance gradually increased from about 20 blows per foot near the ground surface to about 60 blows per foot at a depth of 140 ft. The resistance then increased rapidly as the pile penetrated through the till layer to bedrock.

Deflections were measured in 2 piles with an inclinometer. One leg of a 5-in. angle was tack-welded to the pile web and the other to the inside surface of the flange to provide a duct for the inclinometer casing. The duct extended to within 10 ft of the pile tip. The inclinometer readings (Fig. 11) show both piles to be severely bent around the weak axis, with each pile having a minimum radius of curvature of less than 200 ft. This places the stresses in the flange area well above the yield point.

The load test results of one of these bent piles (14BP73) are shown in Figure 12. The test results appear to be quite satisfactory. The net settlements are appreciable for an end-bearing pile, but most building codes would consider this pile satisfactory for an allowable load of at least 100 tons.

Despite these favorable load test results, the bent piles were considered to be unacceptable for the support of the permanent structure. It was reasoned that under long-term loading, the highly stressed soil supporting the pile in the vicinity of the bent section would consolidate and that this would cause the pile to settle For this reason the construction piles were driven into preaugered holes in an effort to minimize bending. The results of a load test on a steel H-pile (12BP74) that was driven into an 80-ft-long preaugered hole are also shown in Figure 12. This test is much more satisfactory. The gross settlements are similar to those of the bent pile, but this is because the preaugering has removed the surrounding soil to a depth of 80 ft. Note that the net settlements are very small; at 200 tons the net settlement is only 0.06 This pile can be accepted within. out qualification for a design load of at least 100 tons.



Figure 10. Geologic profile at Lambton generating station (10).

As part of the pile-testing program for the Columbia Lock and Dam on the Ouachita River in Louisiana, the Corps of Engineers drove, load-tested, and extracted a pair of 14-in. steel H-piles (14BP73) that were instrumented with strain gages (15).

The piles were driven through 17 ft of stiff fat clays and 14 ft of very dense sand into a tertiary deposit of stiff clay mixed with layers of silty sand. Spoon blows in the tertiary deposit ranged from 60 to 90. Pile 2 was driven with a differential-acting hammer with a rated energy of 36,000 ft-lb. It penetrated to a depth of 51 ft, where it met refusal. Pile 3 was driven with a single-acting hammer rated at 48,000 ft-lb per blow. It penetrated about 30 ft deeper than Pile 2, but below 48 ft the driving was very hard, with driving resistances that ranged from 100 to 700 blows per foot. A total of 479 blows was required to drive the final 7 in.

The strain gages were read during driving and at the completion of the driving. A record was thus attained of both the peak dynamic strains during driving and the residual strains after driving. For Pile 3, the measured peak dynamic strains during driving are shown in Figure 13. The corresponding peak dynamic stresses were 26,100 psi for Pile 3 and 21,500 psi for Pile 2. For Pile 2 the measured residual strains were small. For Pile 3, however, strain gages mounted on the pile web recorded a residual strain of 3,000 microinches at a point about 25 ft above the pile tip. This is well above the yield point, and it was thought that the pile was bent.

Following the completion of the load tests the piles were pulled. The results are shown in Figure 14. Pile 2 was undamaged, but Pile 3 was badly bent, the tip being 6.1 ft from the axis of the straight portion of the pile. The strain gage measurements



Figure 11. Measured deflections of bent H-piles (10).



Figure 13. Peak dynamic strains during driving for Pile 3 (15).

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obtained during the load test (Fig. 15), showed that the bent section of the pile carried a load of 200 tons.

The load test results are shown in Figure 16. The test results on the bent pile are quite satisfactory, but for Pile 2 the gross and tip settlements are both smaller. The load test results for the bent pile are also very similar to those for a pile that was load-tested during the construction of the lock. The construction piles were driven

Pite 2



Figure 15. Distribution of load in Pile 3 (15).

Figure 14. Piles 2 and 3 after extraction (15).

to a depth of only 61 ft in an effort to avoid the severe bending that occurred with Pile 3.

Other load tests on bent piles have

been performed by Parsons and Wilson (<u>16</u>) and Mohr (<u>17</u>). These tests also show that bent piles have considerable load-carrying capacity.



Figure 16. Load-settlement for Piles 2 and 3 (15).

ALLOWABLE STRESSES

The theoretical basis for determining allowable pile loads considers the pile as a column in an elastic medium. Experience has shown that elastic buckling does not occur for piles that are fully embedded in the ground, provided they are not unusually slender. The only reports of initially straight piles that have experienced elastic buckling are very slender piles in very soft soils. Bjerrum (9), for example, reports 2 train rail piles that buckled during underpinning operations on a church in Norway. The section modulus of these piles was only 8.4 in.⁴, and the applied stress at failure was more than 40,000 psi.

On the basis of experience the normal practice is to consider piles as short columns. By this is meant a column that is sufficiently stiff to be designed on the basis of the yield stress of the pile material, rather than the elastic buckling load. The danger of inelastic buckling due to bends and other misalignments is only considered in a general way by placing tolerances on the straightness and verticality of piles that can be inspected and in the selection of allowable stresses.

The allowable stress for cast-in-place concrete piles and reinforced concrete piles is taken as $0.225 f'_{\rm C}$ in most building codes. One notable exception is the Chicago building code, which uses $0.40 f'_{\rm C}$. Timber piles are not stress-graded, and the allowable stress is most often taken as 60 percent of the basic compressive stress for clear material of like species as set forth in ASTM standards. The more liberal codes, such as the BOCA code (1963) and the St. Louis building code (1961), use 100 percent of the basic compressive stress and design as short columns (18).

For some reason there is even greater variety in the allowable stresses used for steel piles. As shown by data given in Table 1, the allowable stress for a steel H-pile in Boston is only 7,500 psi, while the same pile is permitted a stress of 17,000 psi in many cities and a few building codes permit a stress of 20,000 psi. A survey of 80 governmental agencies by the National Academy of Sciences (19) shows an even greater divergence in the allowable stresses used in practice.

To some extent this disparity in the allowable stresses used in practice reflects differences in local foundation conditions and local experiences, but mainly the differences can be attributed to the lack of knowledge of the structural behavior of piles.

The data given in Table 1 show another curious inconsistency in current practice. In most codes steel H-piles are permitted the same allowable stress as steel pipe piles, despite the fact that pipe piles are inspected after driving to investigate for bending and tip damage.

Building Code	Pipe Piles	H-Piles	Remarks
BOCA (1963), Scranton, and Newark	0.50 Fy	20,000 psi	Designed as short column (BOCA)
St. Louis (1961)	0.50 F _v	17,000 psi	Designed as short column
Detroit, Hartford, Prov-	5		-
idence, and Camden		17,000 psi	
New York (1968)	0.35 Fy	0.35 Fy	F _y maximum = 36,000 psi 0.125-in. minimum for pipe
			0.40-in. minimum for H-piles
National (1967)	0.35 Fy	0.35 Fy	0.1-in. minimum for pipe 0.375-in. minimum for H-piles
Chicago (1963) and			
Buffalo (1965)	12.000 psi	12.000 psi	
Atlanta, Seattle, Kansas	, ,	,	
City, Philadelphia, and			
San Francisco		12,000 psi	
Denver (1962)	9.000 psi	9,000 psi	
Boston (1962)	8.500 psi	7.500 psi	
Baltimore (1955)	7.500 psi	8,000 psi	

TABLE 1 ALLOWABLE STRESSES IN STEEL PILES

Note: Data from Johnson and Kavanagh (18).

CONCLUSIONS

In most building codes the selection of allowable stresses for piles is based on column formulas, the more liberal codes treating piles as short columns. For many situations this approach underestimates the ultimate capacity of initially straight piles because it fails to consider that the soil may provide lateral support that increases the critical buckling load (inelastic) beyond that of a short column. This contention seems to be supported by a small amount of field data that indicate that damaged piles have remarkable load-carrying capacity. It appears that in these cases the soil adjacent to the damaged section of the pile had sufficient strength both to support a portion of the pile load and to provide the lateral support necessary to prevent collapse.

Damaged piling, however, may have reduced load-carrying capacity and stiffness. In addition, there is evidence that driving records and load tests are not reliable indicators of pile damage. Long-term load tests and tests with cyclic loading will sometimes indicate pile damage when compared to load tests on undamaged piles.

The published reports of structural pile failures suggest that there are situations where current practice may be underestimating the actual pile loads. Dragdown loading, for example, is sometimes underestimated or ignored. This can be a particular problem in soft, sensitive clays where remolding due to pile driving can cause dragdown loading to develop with time. Bjerrum reports this phenomenon in the very sensitive Norwegian clays (9).

What is required to put the structural design of piles on a truly rational basis is more good field data, particularly research into the basic causes of pile damage and the loadcarrying characteristics of damaged piling. With sufficient data it should be possible to develop a rationale that would permit the use of higher stresses for favorable foundation conditions, with the more conservative values being reserved for difficult situations. This type of approach is presently used in Oslo, Norway, where a stress of 14,200 psi is permitted for steel H-piles less than 40 ft long, 11,400 psi for piles 40 to 100 ft long, and only 9,200 psi for piles over 100 ft long (9).

Situations where higher stresses could be considered include piles that are inspected for damage after driving, piles embedded in firm soils that are capable of providing substantial lateral support, piles that cannot be subjected to dragdown loading, and piles whose support is primarily derived from friction. It also seems that in many cases some benefit could be taken for the load carried by the base reaction of the pile cap.

The selection of allowable pile stresses should also include consideration of the maximum stresses during driving. The use of the wave equation (20, 21) looks encouraging as a method for predicting driving stresses; however, no comparison with field measurements has yet been reported. To accurately predict pile behavior during driving will require an investigation of the effects of dynamic stress on pile materials.

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