Low-cycle fatigue strength of steel piles

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Summary

In integral abutment bridges clamped abutment piles are in addition to a compressive normal force subjected to bending load cycles from daily and yearly temperature variations. Through experiments with full-scale specimens representing a clamped pile it is shown that a steel pipe pile loaded in bending can withstand several hundred load cycles at strain ranges greater than 6 times the yield strain with almost full load bearing capacity. By means of an example it is shown that by permitting pile strains greater than the yield strain, in contrast to most present design codes, integral abutment bridges can be erected with a span length up to 500m and a prospected service life of 120 years

Keywords: Integral abutments, low-cycle fatigue, steel piles

1. Introduction

Steel piles in integral abutments bridges may in service be severely strained due to soil restraint and yearly bridge movements which enforce displacements and rotations to the pile end that is clamped to the superstructure see Fig. 1.

The bending strains in the piles are not necessary for the bridge to soil load transfer and would of course virtually vanish in a hinged pile, but a mechanism of this kind is both more expensive and susceptible of frequent maintenance problems. A pile joint exempt of a hinge mechanism is therefore preferable in practice. A pertinent question is to what extent, if any, repeated inelastic strains might affect the load carrying capacity of a pile. Is it possible to altogether ignore such secondary effects and still retain a safe design?

Most design codes do not allow strains exceeding the yield point in the serviceability limit state. Besides a general fear of plasticization, concern for low cycle fatigue is believed to underlie this conservatism. In the ultimate limit state, however plasticization of the pile cross-section is not unconditionally out-ruled in present codes.

The ability of a pile to carry both normal force and imposed deformations at the top is supported by full scale static tests with X-shaped steel piles [1]. The aim of the new tests carried out at Lulea University of Technology, Sweden, is to demonstrate that steel pipe piles in fact are capable to withstand cyclic strains much greater than the yield strain during several hundred cycles and still safely carry the loads needed. A bridge is normally designed to be in service for 120 years in Sweden.

Some low cycle fatigue tests are described in literature. Dusicka et al. [2] tested five different structural steels with static yield strength between 100 and 485 MPa under cyclic plastic uni-axial straining in the range $\pm 1\%$ to $\pm 7\%$ strain amplitude. Cyclic hardening was observed in all steels and the maximum cyclic stress was found to be dependent on the steel type and the strain range. They also found that steel types of similar manufacturing specifications are characterised by rather similar cyclic stress-strain curves when subjected to cyclic inelastic strain, irrespective of the monotonic yield strength and that the overall fatigue life was similar for each of the steels tested.

Fig. 1: An integral abutment.

Tateishi et. al. [3] have built a unique bend testing machine for low cycle fatigue tests in order to obtain very large cyclic strains. Their very low cyclic fatigue life results for a structural steel are in agreement with Dusicka et al. [2]. It was also concluded that the fatigue life predicted by Palmgren-Miners rule together with Manson-Coffin's relationship grossly overestimates the actual life in extremely low cycle fatigue.

2. Low cycle fatigue tests

2.1 Material

The material in the piles used in the tests is a fine grain carbon-manganese structural steel with designation S440J2H. Typical carbon equivalent is $CEV = 0.30$ (maximum 0.39). The chemical composition is shown in Table 1. The measured diameter of the pipe was 113.9 mm, and the measured thickness 8.00 mm.

Table 1: Chemical composition max weight percent.[4].

	C	Si	Mn	P	S
Maximum	0.18	0.25	1.60	0.020	0.018
Typical	0.07	0.18	1.4	0.010	0.006

Mean values from three static uni-axial tensile tests are yield strength $f_{y0.2}=480$ MPa and ultimate strength $f_u=520$ MPa and maximum elongation $\epsilon_{max}=23\%$.

2.2 Testing of pipes

2.2.1 Test set-up

Fig. 3: Test set-up.

of the gauges measured the longitudinal strains on the top of the pipe at distances 100 mm, 200 mm and 300 mm from the concrete block end. Two of the gauges measured both longitudinal and

Fig. 2 Test arrangement.

The test specimens were made of a steel pipe of 2100 mm length cast into a reinforced concrete block of 600 mm length and a square 200 mm cross section. The concrete were centred about the mid-point of the pipe, Fig. 2. The steel pipes are cold formed and have a longitudinal weld. The pile was oriented so that the weld was placed in the neutral axis during the tests. Each pipe was equipped with five strain gauges. Three of the gauges measured the longitudinal strains on the top of the pipe at distances 100 mm, 200 mm and 300 mm from the concrete block end. Two of the gauges measured both longitudinal and transverse strains at a distance 10 mm from the concrete block end both at the bottom and the top of the pipe cross section. The deflection of the load point (at one pipe end) was measured with a displacement gauge as well as the unloaded end of the specimen. The accuracy of the displacement gauges used is within 0.1%. The load (or displacement) was applied 700 mm from a near end of the concrete block through a servo hydraulic cylinder (Instron 270 kN) and a 250 kN load cell.

Two specimens were prepared and each specimen was used in two tests. A steel beam was placed along and on top of the concrete block supported by the loading frame floor. A second steel beam was placed perpendicular to and on top of the first beam and anchored to the frame floor with three 32 mm diameter Dywidag bars. The bars were pre-stressed to prevent a concrete block from moving and to keep the block in tight floor contact during a test. The resulting force acting on the concrete

block due to pre-stressing was 900 kN. The test arrangement is shown in Fig 3.

2.2.2 Testing procedure

The tests were started by slowly increasing the load on the pipe in the upward direction until a plateau of the load-displacement curve was approached. This was achieved at a load level of 100 kN for all tests and corresponds very well with the theoretical plastic moment capacity of the cross section. The load was then reversed until a similar plateau was reached for downward loading. The plateau load for downward loading was -95 kN, i.e. a somewhat smaller value than for the upward load. The measured displacements corresponding to the plateau loads were then used to control the displacement at the loading point of the pipe. The loading point displacement was applied cyclically in the range from 26.7 mm to -26.6 mm, for the first specimen. For the remaining three tests the corresponding pipe end displacement are shown in Table 3. The loading frequency was always kept at 0,5 Hz.

Table 3: The table shows the limits between which the imposed deformation was cycled between during the pile tests

Test number	Upward displacement (mm)	Downward displacement (mm)
1	26.7	-26.6
2	22.5	-23.5
3	20.6	-17.5
4	24.2	-22.8

2.2.3 Results

Fig. 4: The cross section of the cracked pipe after test.

The failure mode was low cycle fatigue cracking in the steel pipe as shown in Fig. 4, for all four tests. The cracks initiated at the top of the pipe cross-section close to the end of the concrete block and grew initially through the pipe wall. The cracks were first observed at 711, 319, 366 and 402 cycles, respectively, in the four tests.

Cyclic softening was observed during all tests, as can be seen in Fig. 5, which shows the vertical load applied on a pipe as a function of time in a test. The softening rate was greatest during the first 200 load cycles of all tests and decreased thereafter. The softening behaviour may be due to softening of the steel material itself or/and softening of the concrete around the pipe.

The first test was halted at 145 cycles as it was observed that the loaded part of the pipe was slowly but steadily moving out of the concrete block in each load cycle. The top and bottom strain gauges 10 mm (see Fig. 2.) from the concrete block end in this test

Fig. 5: Force as a function of time during test.

were destroyed and could thus not be used during the later part of the test. After this incident the position of the remaining pipes were fixed longitudinally by welding a small steel tab to the pipe wall, flush against the concrete.

Following crack initiation and early growth, after several hundred load cycles there is a rapid drop of the load bearing capacity of a pile during some 50 load cycles as the wall-through crack propagates in two opposite directions, each along the circumference of the pipe cross-section. At the end of a test the crack has

propagated over some 60% of the circumference of the cross-section. The residual load bearing capacity is not symmetric with respect to the direction of the load. For a load closing the crack the residual load bearing capacity is still some 40% of that of an undamaged pipe. This implies that the crack is not closing ideally which in turn indicates significant plastic deformation around the crack tips. For a crack opening load, on the other hand, the residual load bearing capacity is almost reduced to zero.

3. Analysis and discussion

The fatigue resistance of a steel structure is often assessed according to EN 1993-1-9 [5]. A relevant detail category, $\Delta\sigma_C$, equal to the fatigue strength at 2 million cycles, is selected from the detail library. The number of constant amplitude cycles to failure N_f at the nominal stress range $\Delta\sigma$ can be calculated with the formula:

$$N_f = \frac{\Delta\sigma_C^m}{\Delta\sigma^m} \cdot 2 \cdot 10^6 \quad (1)$$

where $m = 3$ in the range of interest here. For pipe base material $\Delta\sigma_C = 160$ MPa (the longitudinal seam weld of the pipe is in the neutral axis). The nominal elastic stress in

Eq. (1) is a uni-axial nominal stress and can be converted to a nominal strain

$$\Delta\varepsilon = \Delta\sigma / E \quad (2)$$

where $E = 210$ GPa is the elastic modulus. Eq. (1) after substitution of Eq. (2) can now be extrapolated to include plastic strains. Eq. (1) in terms of plastic strains is plotted in Fig. 6 and compared with the test results.

The values obtained with expression (1) are design values while the test results are values at failure. To obtain design values from test results an expression similar to (3) is often used [6]

$$X_k = m_d \cdot (1 - k_n \cdot V_x) \quad (3)$$

where m_d is the mean value obtained from tests, k_n is a coefficient depending on the number of tests and the target safety level, V_x is the coefficient of variation and X_k is the characteristic value that is used for design. The coefficient k_n can be obtained from EN 1990:2002 [6] and varies between 1.64 and 3.37 depending on the number of tests and whether the coefficient of variation is known. The variation coefficient V_x of fatigue life varies in a wide range and is dependent on material, test procedures; stress level and whether specimen is smooth or notched [7]. If it is assumed that the number of cycles to failure at tests has a coefficient of variation in the range between 0.05 and 0.2 the mean number of cycles from tests shall be divided by a factor of between 1.1 and 3.1 to get a design value. In Figure 6 two lines have been drawn. One curve representing the design curve and one where the number of cycles has been multiplied by 2 to be comparable to the test values. The test values seem to agree well with the upper curve but the inclination differs.

The number of cycles to failure in low cycle fatigue is often described with the Coffin-Manson relationship

$$\frac{\Delta\varepsilon_{eq}}{2} = \frac{\sigma'_f}{E} \cdot (2N_f)^b + \varepsilon'_f \cdot (2N_f)^c \quad (4)$$

Fig. 6: Test results compared with design values according to EN 1993-1-9 [5].

The first part of the equation is dominant at elastic strain levels and the second at plastic strain levels. The coefficients and the exponents in the expression are material dependent and many experiments have been carried out to calibrate them for different materials.

In Fig 7 the curves for expression (2) are plotted for different steels, taken from [8], [9] and [2] and together with the present test results. The values from [2] are taken directly from the paper whilst [8] and [9] are calculated with expression (2) and the values given in Table 4. Values are also calculated for $\Delta\sigma_c = 160$ MPa with expressions (1) and (2) as described above. The pipe tests have lower values compared to tests by the other authors. This can be explained by the fact that the pipe test strains are bi-axial while the other tests are uni-axial. Another explanation is that the strain gauges used failed prematurely and that the strains varied during the test and could not be recorded during the whole test.

Fig. 7: Curves for expression (4) drawn for different steels.

Table 4: Fatigue parameters for different steel materials according to Chakraborti et. al [8] and Sewerynet et. al. [9].

	RN [8]	SQ [8]	FP [8]	Tube steel LG290GA [9]	SAE 1095 [9]
σ_f'	1300	1340	940	948	547
b	-0.078	-0.090	-0.093	-0.092	-0.0189
ε_f'	0.129	0.150	0.65	0.26	0.7163
c	0.455	0.514	0.624	-0.445	-0.6391

The first strain cycles, measured with the gauges 10 mm from the concrete edge, for the first test are shown in Fig 8. The strain varies in compression and tension between $-13500 \mu\text{strain}$ to $2400 \mu\text{strain}$. The strain range shown represents just the first cycles during the tests

Fig. 8: The figure shows force and strains for the first during the first test.

Table 5: Strain range measured with gauges 10 mm from concrete end (μ strains).

	Test 1	Test 2	Test 3	Test 4
$\Delta\varepsilon_{eq}$	15 900	No data	16 300	16 500

because this strain gauge failed prematurely so that a mean strain range up to failure could not be recorded. Strain ranges for the other tests are shown in Table 5.

4. Example

The cyclic strains in piles supporting an integral abutment bridge are caused by daily temperature variations superimposed on greater yearly temperature variations and by traffic. The length of the bridge changes with the temperature variations and the abutment including the clamped pile end is displaced. When loaded, the superstructure rotates around the neutral axis at the abutments, forcing the piles to accommodate the rotation. The rotation also gives extra horizontal displacement of the pile connection, increasing with the height of the abutment. By means of an example is shown the benefit of allowing strains exceeding the yield point in the abutment piles.

Fig. 9: The bridge over the creek Leduån.

The bridge in the example is a 40 m single span integral abutment bridge. The composite superstructure comprises two H-shaped welded steel beams, and a one lane slab of concrete with characteristic compressive strength $f_{ck} = 40$ MPa. The bridge, spanning the creek of Leduån, is built in reality and has also been used for monitoring the stresses in the piles and the steel girders, during 18 months. The monitoring gave lower strain ranges than the calculations presented here and also than FEM calculations taking the stiffness of the soil into account. The reasons for this were lower temperature in the bridge than in the calculations, but also softer behaviour of the soil and possibly a non-stiff connection steel-concrete at the pile top.

Table 6: Data for calculating strains in pile.

$\varnothing$	0.1635	m
A_{pile}	3722.3	mm ²
H	1.65	m
$L_{equ.m}$	4.247	m
$L_{equ.h}$	4.203	m

The superstructure is supported by six end bearing piles, of steel pipe RR170x10, in a single row under each abutment, driven into the soil along a straight line perpendicular to the long axis of the bridge. The bearing piles were sheltered to a depth of 2 m under the lower end of an abutment by wider steel pipes, of diameter 600 mm. Styrofoam plates were inserted inside the sheltering pipes and the remaining space was filled with loose sand. A detailed description is given in Hällmark [10] or Nilsson[11]. The

results of calculations by Hällmark [10], according to the Swedish Bridge Code, are summarised in Table 7 and the input data used are shown in Table 6.

The resulting strains are calculated with expression (5) and the model used for the calculations is shown in Fig. 9. The maximum calculated strain is 1634 μ strains or 343 MPa in terms of stress. The utility rate is then 97% for S355 steel. That means that the length limit is almost reached for this bridge if strains exceeding the yield point are excluded. The strains in the piles from the normal force, which is the only load transferred to the ground by the piles, is only 25 % of the total strain in this example (414 of 1634 μ strains, see Table 5).

Fig. 10: The model used for the calculations in this example.

The total strain range in this example is 1353 (= 1634 – 282) μ strains corresponding to a stress range of 0.8 f_{yk} . The adoption of a strain range $\pm 2f_{yk}/E$ implies that we could have a 5 times larger strain range. Assume that the strain range due to traffic remain the same if the bridge is made longer, due to the facts that we

Table 7: Loads, rotations and displacement acting on pile top.

	ε (μ strain)						
	θ (%)	Δ (mm)	N (kN)	Rotation	Displacement	From force	Total
Steel girders	0	0	34	0	0	43	43
Concrete deck	0	0	125	0	0	160	160
Pavement	0.148	0	38	114	68	49	231
Crash barrier	0.006	0	2	5	3	2	10
Shrinkage	0.248	0	0	191	114	0	305
Total permanent action	0.402		199	310	184	255	748
Temp. Var. (winter)	0	-10.6	0	0	-294	0	-294
Temp. Var. (summer)	0	4.9	0	0	136	0	136
Temp. Grad (15)	-0.140	0	0	-108	-64	0	-172
Temp. Grad (-18)	0.169	0	0	130	77	0	207
Traffic Max norm. Fatigue	0.108	0	200	83	49	256	388
Traffic Max mom. Fatigue	0.317	0	125	241	143	159	543
Total maximum	0.883	4.9	324	680	541	414	1634
Total minimum	0.262	-10.6	199	201	-174	255	282
Passive earth pressure	-0.151	0	0	-116	-69	0	-185

make the girders larger and also put in more supports. This means that the strain range could be increased with $4 \cdot 1353 = 5412$ μ strains.

As the bridge length 40 m gives a strain range of $294 + 136 = 430$ μ strains, the length could be increased with $40 \cdot (5412/430) = 500$ m. Other criteria than the fatigue criterion also have to be checked, and different load factors in other codes might lead to slightly different results, but the calculations based on the pile testing indicate that a bridge length up to 500 m seems possible, with regard to fatigue due to pile straining.

$$\varepsilon = \frac{N}{A_{pile} \cdot E} + \left(\frac{3 \cdot \Delta}{L_{equ,h}^2} \right) \cdot (\Delta_{abut} + H \cdot \theta) + \left(\frac{2 \cdot \Delta}{L_{equ,m}} \right) \cdot \theta \quad (5)$$

5. Conclusion

The abutment piles in integral abutment bridges are almost always subjected to a large number of bending load cycles from daily traffic and daily and yearly temperature variations in addition to a compressive normal force. In the present work it has been demonstrated experimentally that a steel pipe pile loaded in bending can withstand several hundred of cycles at strain ranges greater than 6 times the yield strain. By means of an example it is further shown that by permitting pile strains exceeding the yield strain (in contrast to most present design codes), integral abutment bridges could be erected with a span length up to 500 m. The tests described in this paper is a first step with encouraging results towards the development of future simple and safe design rules for integral abutment bridge pile capacity. Furthermore, fatigue tests of coupons from the same steel pipes have been carried out at different strain levels, and will be reported later.

6. References

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