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Long-Term Monitoring of Strain in Instrumented Piles

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Abstract: The development of strain in two 31 and 56 m long instrumented postdriving grouted cylinder piles at a site west of Busan, South Korea, were monitored during 200 days after construction, whereupon a static loading test was performed. Initial strain measurements showed unexpected elongation of the pile, probably due to swelling from absorption of water but as the soil reconsolidated, the elongation changed into shortening, probably due to imposed residual load in the pile. The resulting compression of the pile eventually offset the swelling of the pile. To investigate the cause of the strain changes more closely and enhance the evaluation of the field data, two short pile pieces were prepared and placed free-standing above ground in an outside laboratory. One piece was from a cylinder pile of which central void was grouted and one was made up by grouting inside a temporary casing. The monitoring showed that the short pieces appeared first to shorten and then to elongate due to the heating and cooling from the hydration process. When strains and temperature had stabilized 150 days after start of the study, both pieces were submerged to introduce swelling of the concrete. For the first 100 days after submerging, the swelling strains in both short pieces amounted to 100 $\mu\varepsilon$. Seven hundred days after submersion, the total swelling strains were 150 and 250 $\mu\varepsilon$.

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CE Database subject headings: Instrumentation; Piles; Grouting; Strain; Measurements; Swelling; Monitoring.

Introduction

The Nakdong River estuary delta covers a large area west of the city of Busan, South Korea, and is composed of very thick deposits of compressible normally consolidated clays with occasional interbedded sand layers. Most of the land has been lying vacant due to high construction costs of the deep foundations necessary for all but light structures. Recently, however, because of space becoming increasingly limited in the city, heavy buildings are being designed and built in the area. The design has to take into account consolidation settlement caused by fill placed to raise the land above flood levels. For this reason and because of the low bearing capacity of the deltaic soils, all the buildings require piled foundations. The commonly employed deep foundation system in Korea consists of steel pipe piles driven to competent soil layers, such as bedrock or dense soil. Where these piles then become very long, such as the 70 to 80-m lengths anticipated for the Nakdong River estuary delta, the foundations become very costly. For the development of two areas, called Shinho and Myeongji, to be developed with tall apartment buildings to house approximately 80,000 people, an alternative pile type is considered: a closed-toe cylindrical concrete pile locally called pretensioned spun high strength concrete (PHC) piles (in North America, a very similar pile is known as the ICP pile). The costs of a piled foundation supported on the PHC pile are between a quarter to a third of those of a foundation supported on the steel piles. The PHC pile is considered less suited for the hard driving required to reach the termination depths usually selected for the steel piles. However, it was thought possible to limit the pile installation depth at the Shinho and Myeongji sites by terminating the piles in an intermediate sand layer. Prior to the design of the piled foundations, therefore, a full-scale testing program was executed involving drivability and load response of three strain-gauge instrumented test piles. The instrumentation consisted of pairs of vibrating wire strain-gauges placed at different depths in the central void in the pile. The void was then grouted. The purpose of the instrumentation was to determine the load distribution in the pile in static loading tests performed about 6 months after the end of driving, when the soils were expected to have reconsolidated ("setup").

The strain-gauge records taken during the static loading test can be converted to loads to establish the distribution of the loads imposed in the test. However, prior to the start of the loading test, loads will develop in the pile due to reconsolidation of the soil after the driving. These loads are called locked-in loads or residual loads. For the subject project, in order to determine the magnitude and distribution of the residual loads, the strain-gauges were read frequently after the grouting of the central void. It was then found that the build-up of strain recorded by the gauges showed some unexpected behavior. A laboratory study was therefore instigated consisting of two 2.0-m-long strain-gauge instrumented pieces of pile where no soil could influence the measured strains, leaving only the effects of the curing of the grout and, as it turned out, of temperature variations. One of the short pieces is taken from a PHC pile and one is a dummy cast in an axially flexible casting mold. This paper reports and discusses the strains observed in the test piles and short pile pieces.

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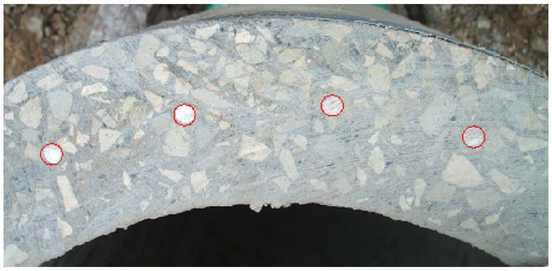


Fig. 1. Photo of the PHC cylinder pile cross section. Four prestressing rebars are circled in red.

Observations in Full-Scale Test Piles

The PHC test pile used at the site is a 600-mm outside diameter prestressed concrete cylinder with an 85-mm wall (cross section area $=1{,}375\ \text{cm}^2$) embedding 24 9.2-mm rebars (steel area to concrete area ratio of 1.1%) anchored to steel end plates with a 430-mm central hole. The net prestress is about 8 MPa. The pile is cast in 5 to 15-m segments. The segments are spliced by welding the end plates together in the field, one splice at a time as the pile is driven. The concrete consists of Portland cement and concrete aggregates of crushed granite. The nominal cube strength is 80 MPa. Fig. 1 shows a photograph of a portion of the pile cross section.

The testing program included two PHC test piles, one at the Shinho site and one at the Myeongji site. The Shinho pile (56-m embedment) was driven on January 18, 2006 and the Myeongji pile (31-m embedment) was driven on January 17, 2007. After the driving, a reinforcement cage was placed and grouted in the central void of each pile. The cage consisted of three longitudinal 22-mm rebars and a 10-mm-diameter spiral reinforcement at a 450-mm pitch. For the Shinho pile, 12 pairs of vibrating wire sister-bar gauges (Geokon Model 4911A) were attached to the reinforcing cage in a vertical position before lowering the cage into the central void. The Myeongji pile had seven pairs. The uppermost pair was placed at the ground surface for the Shinho pile (the pile had a 1-m stick-up) and at 4 m below ground in the Myeongji pile. After completed driving, the central void in each pile was filled with Portland cement grout without coarse aggregates. The grout in the Shinho pile had no sand content, the water/cement ratio was 0.45, and the cube strength was 18 MPa. The grout in the Myeongji pile had proportions of water, cement, and sand of 22, 67, and 11%, respectively, and a water/cement ratio of 0.33. The cube strength was not tested. Curing-acceleration and water-reducer admixture, types GeoFix1p and Sika Viscocrete-R, were included for Shinho and Myeongji piles, respectively, to an amount of 1% of the cement weight.

The strain-gauges were read at frequent intervals from their installation into the void in the pile and after grouting the void. Figs. 2–4 show the development of temperature and strain recorded at eight gauge levels in the Shinho O-cell pile. The values shown are from the two gauge pairs (#12) placed 1 m below the pile head (at the ground surface) and from single gauge pairs (#11, #10, #9, #8, and #2) placed at depths of 4, 9, 14, 19, and 49 m, respectively. Intermediate gauge records are omitted for clarity. The uppermost gauge location consisted of two pairs of sister-bar gauges and are marked #12A–12D. Records were taken until the start of static loading tests some 200 days after the grouting. The records beyond the first 40 days are not shown in the figures. Negative values indicate apparent shortening (compression) of the sister-bar strain gauges and positive values indicate apparent lengthening (elongation).

As shown in Fig. 2, the hydration process of the cement grout gave rise to a steep increase of temperature during the first about 15 h after grouting the pile, whereafter the temperature diminished at a progressively smaller rate to a stable temperature after about 10 days.

The recorded strain changes in the rebars are shown in Fig. 3. During the first 15 h, coinciding with the increase of temperature, the vibrating wire readings decreased, that is, the rebars shortened, suggesting increasing compression in the pile. When the temperature reduced, the readings indicated that the rebars first recovered the shortening and then elongated, suggesting that tension developed in the pile. When the temperature had stabilized, which took about 10 days, the measurements indicated a net tension in the pile.

The strain values are not adjusted to the difference in thermal

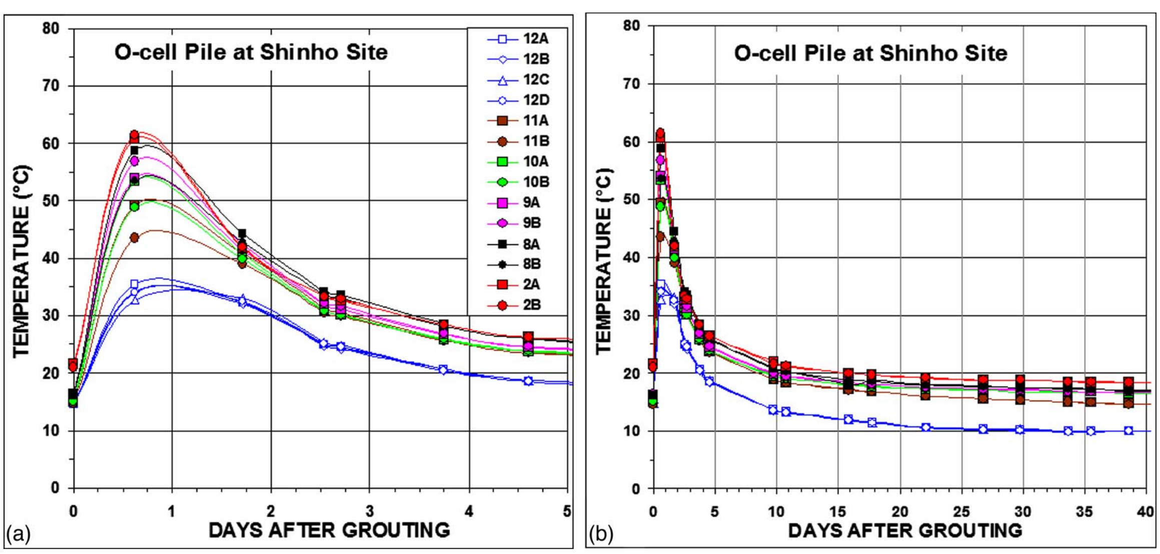


Fig. 2. Development of temperature in the O-cell pile at the Shinho site

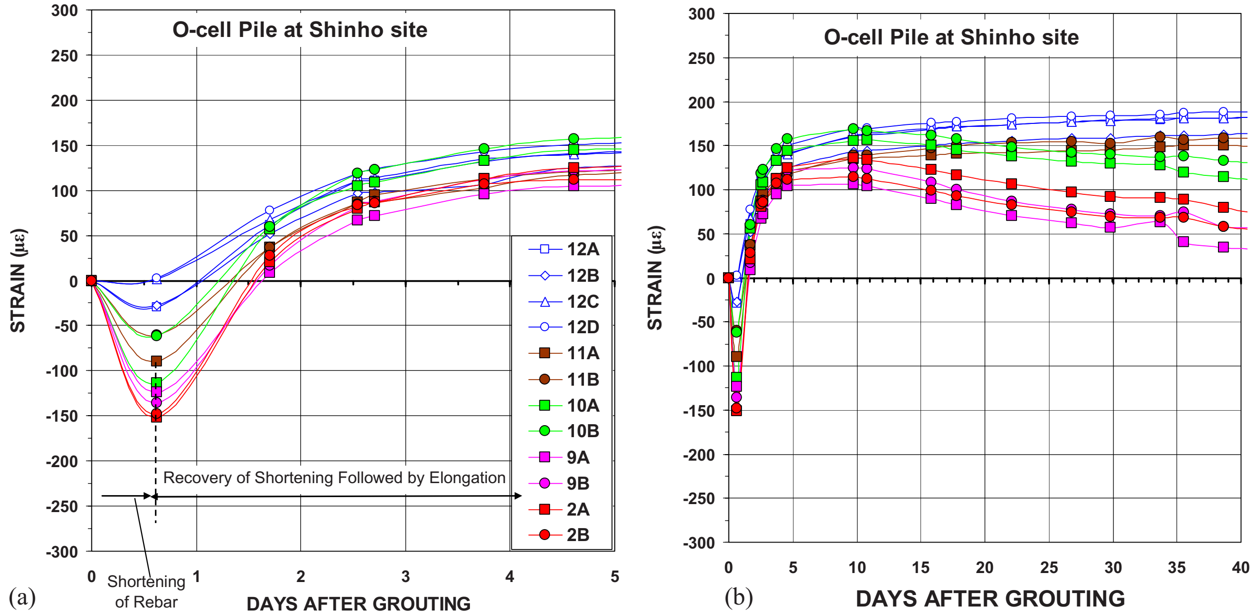


Fig. 3. Development of strain in the O-cell pile at the Shinho site

coefficients between the rebar and the concrete and grout. Adjustment (see discussion below) would have reduced the values somewhat during the first few days, when the temperature kept changing. However, the later elongation—tension trend—of the rebars at constant temperatures would still occur.

It was expected that the gauges down the pile would record increasing compression strain (shortening of the rebar) due to reconsolidation of the soil around the pile after the driving disturbance. However, no such change was expected to occur at the ground surface gauges (#12). Yet, the records of #12 indicate a slight elongation of the rebars over time, that is, implying a tension load developing in the pile. Further down the pile, the records indicate an elongation with time, that is, a reduction of load in the pile with time is suggested. It is hypothesized that the

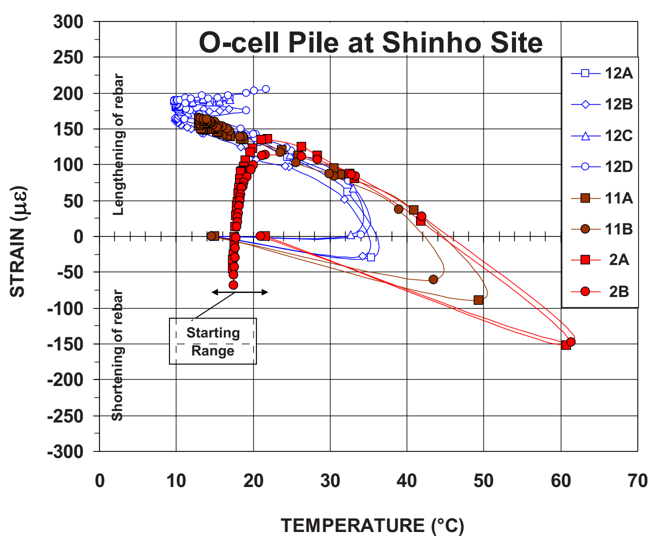


Fig. 4. Development of strain versus temperature in the O-cell pile at the Shinho site

elongation is due to swelling when the concrete absorbs water from the soil.

The temperature dependency is more clearly demonstrated in Fig. 4 showing the strains as a function of the recorded temperature.

Figs. 5–7 show the similar development of temperature and strain recorded by the gauge pairs placed in the O-cell pile at the Myeongji site. The uppermost gauge pair is marked #7A and #7B. The strain values shown are from single gauge pairs (#7, #6, #5, #3, and #1) placed at depths of 4, 9, 14, 24, and 30 m below the ground surface. Intermediate gauge records are omitted for clarity. The trends of temperature strain developments in the Myeongji pile are similar to that of the Shinho pile. However, the recorded temperature and strains are much larger for the Myeongji pile. The difference could be due to the different water/cement ratios of the grout for the piles, 33% as opposed to 45%, respectively.

For comparison, measurements are offered of strain in a bored pile during the curing of the concrete taken in a 74.5-m-deep, 2.6-m-diameter pile constructed in 2006 in the Fraser River deltaic soil, Golden Ears Bridge, Vancouver, British Columbia. A 22-m-long permanent steel casing was installed prior to concreting. The pile was instrumented with 10 levels of vibrating wire strain-gauge pairs type Geokon. The most shallow gauge level was at 9.5-m depth and the deepest was at 66-m depth. No information is available on the composition of the grout. The measurements were taken from the grouting of the pile until the start of a static loading test (multilevel O-cell test), 35 days later. Fig. 8 shows the temperature measurements at the gauge levels. The maximum temperature is approximately the same as for the Shinho pile but it took two to three times as long to reach. At the time of the static loading test, most but not all hydration heat had dissipated. The measured temperatures varied between individual gauges and depths but there was no correlation to depth or to whether a gauge level was inside or below the upper 22-m casing.

The recorded changes of strain in the Golden Ears bored pile

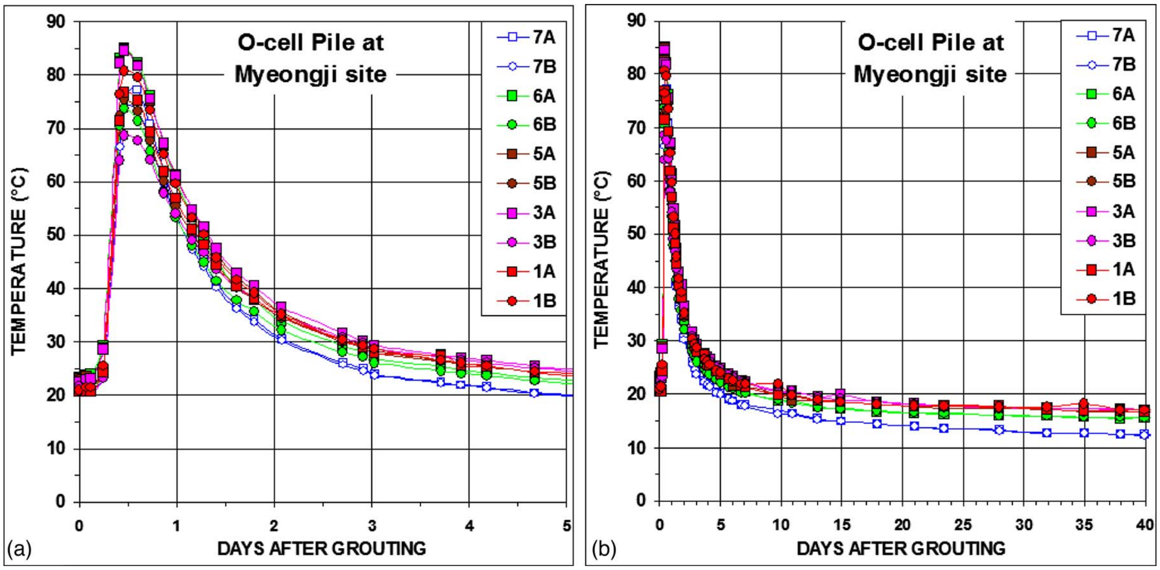


Fig. 5. Development of temperature in the O-cell pile at the Myeongji site

are presented in Fig. 9. All curves shown are from uncased portion of the pile. During the first 5–10 days, the strain measurements show that the rebar shortened. Thereafter, a slight recovery of shortening occurred. There was no net elongation of the rebars.

For the upper gauge levels, the changes of strain recorded in the three piles during the curing period are larger than the maximum residual load that can reasonably be expected to develop in the pile. For the Golden Ears Bridge, for example, the strains imposed during the static loading tests, were much smaller than those shown in Fig. 9. Clearly, in addition to the influence of the soil transferring load to the test piles, the records of the strain-gauge instrumented rebars (sister bars) in the test piles are influenced by curing temperature, swelling due to water absorption of the grout, and, for the PHC piles, the restraining of the expansion and contraction of the grout by the cylinder wall and its absorption of water from the soil and the grout—not a factor for the bored pile.

A laboratory study was undertaken to shed light on the cause

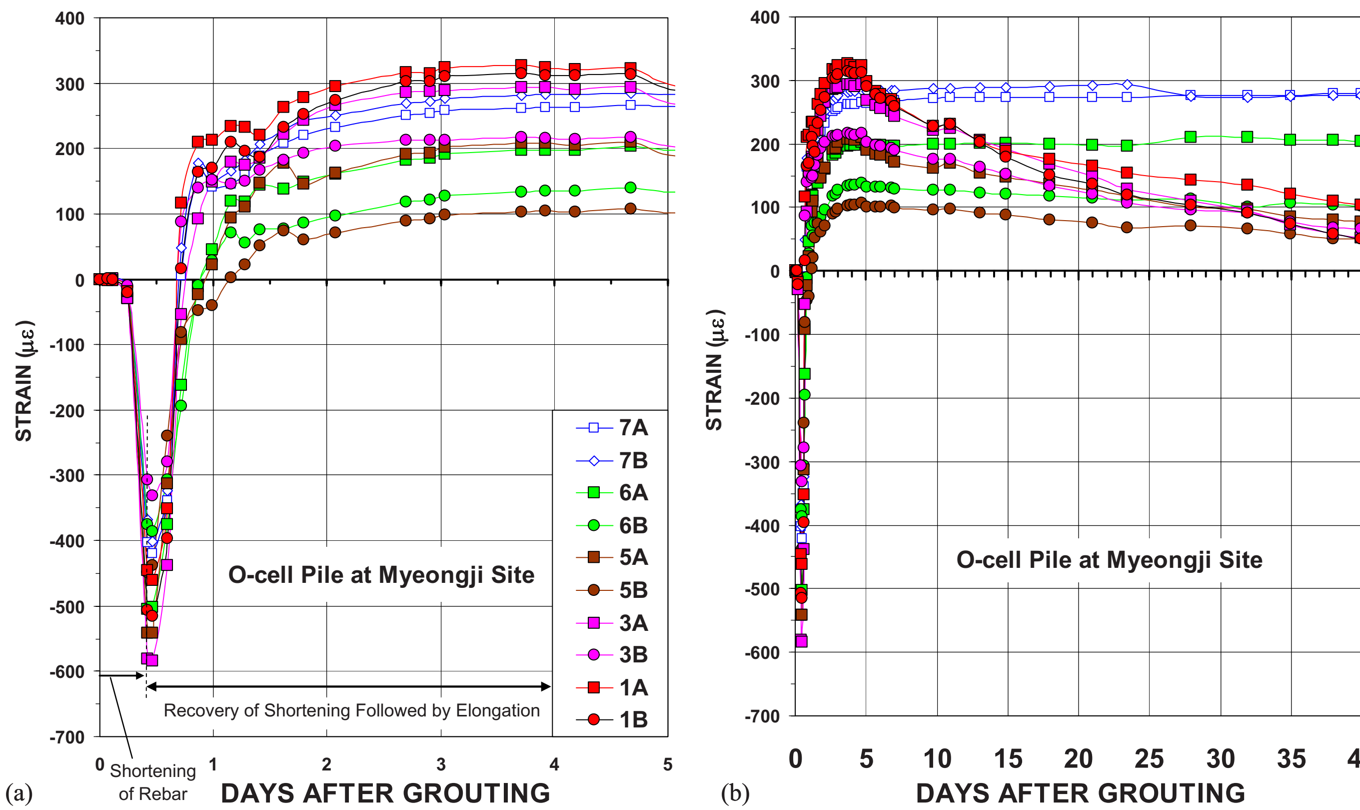


Fig. 6. Development of strain in the O-cell pile at the Myeongji site

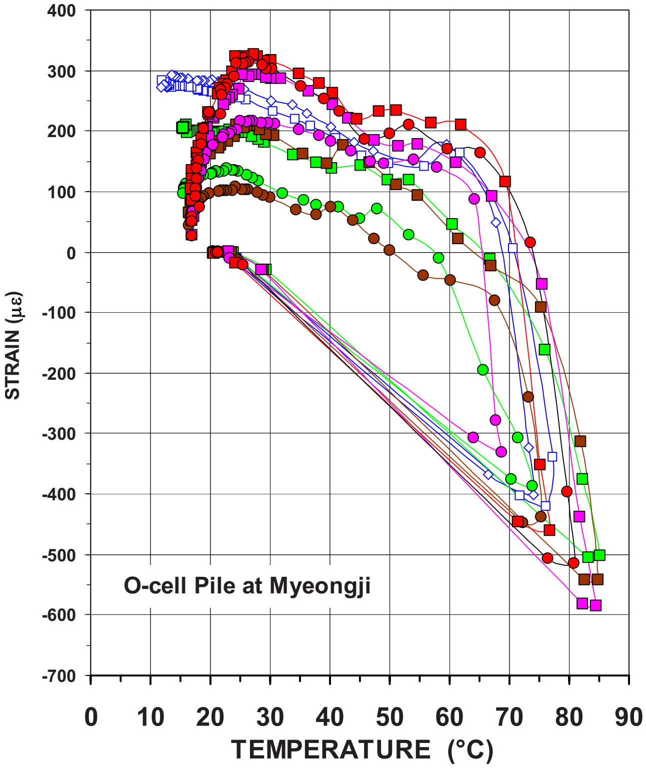


Fig. 7. Development of strain versus temperature in the O-cell pile at the Myeongji site

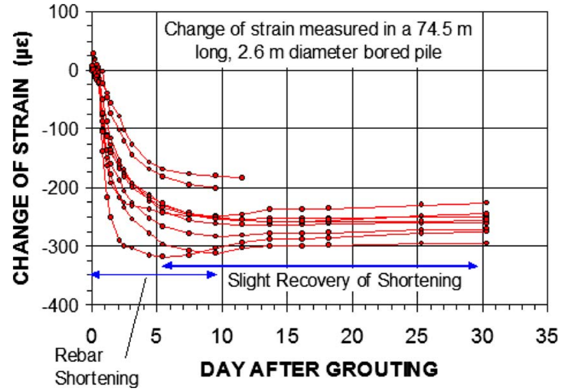


Fig. 9. Change of strain during the curing of grout in the Golden Ears Bridge test pile (data courtesy of Trow Engineering Inc. and Amec Inc., Vancouver, B.C.)

of the strain development and to attempt obtaining means to a numerical evaluation of the strains due to nonsoil related effects.

Laboratory Study

A laboratory test was undertaken wherein the soil influence was removed. The test consisted of two 2.0-m-long pieces of pile. One piece was from a PHC pile and one was a "bored pile," consisting of casting the pile in a mold consisting of a plastic pipe with a 500-mm inside diameter and a wall thickness of 30 mm. The *E*-modulus of the concrete is approximately 30 GPa. In contrast, the stiffness of the plastic pipe is very small—an axial load of a mere 5 kN on the pipe results in 5% axial strain. Therefore, the plastic pipe will impart negligible restraint to axial strains developing in the concrete, as opposed to the high restraint of PHC pile cylinder wall.

The test was set up outdoors. An 800-mm plastic pipe was placed around the two pieces to protect against rapid variation of ambient temperature. The two pieces were instrumented with two pairs of a Korean manufactured vibrating wire strain-gauge which is similar to the Geokon gauge. The gauges were placed in a square configuration attached to the same arrangement reinforcement cage as used for the field piles and grouted. The grout consisted of cement paste without coarse aggregates and a water/cement ratio of 0.45. No hardening accelerator admixture was included.

The laboratory study started on April 25, 2006. Fig. 10 shows a sketch of the short-piece test arrangement.

As indicated in Figs. 11 and 12, the temperature development

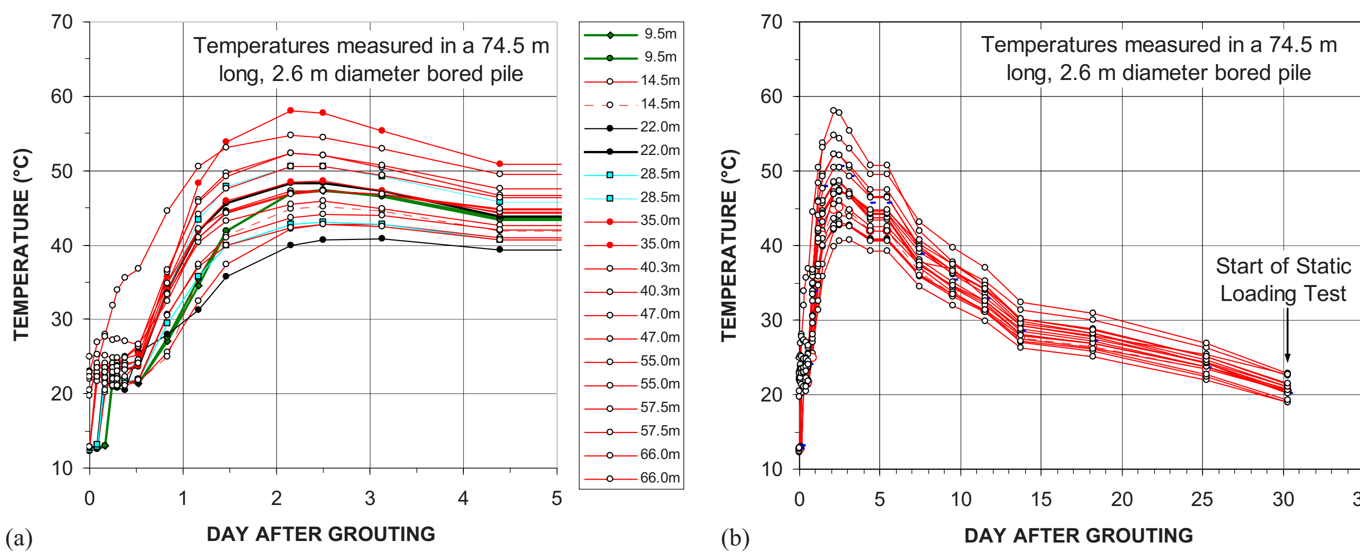


Fig. 8. Temperature records during curing of grout in the Golden Ears Bridge test pile (data courtesy of Trow Engineering Inc. and Amec Inc., Vancouver, B.C.)

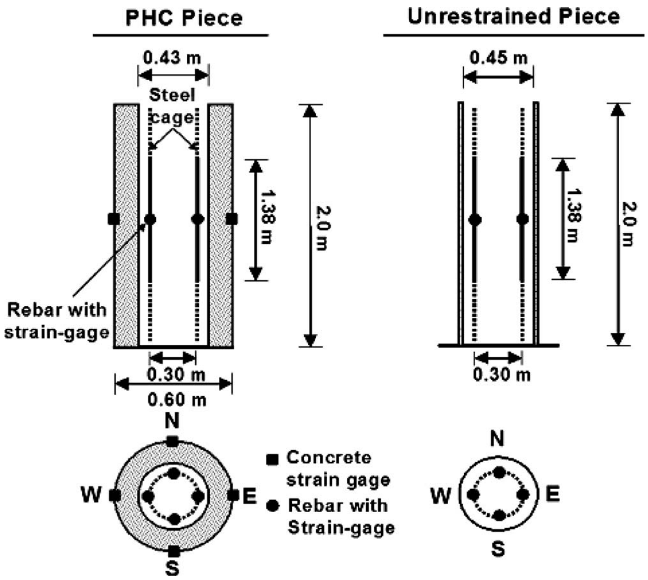


Fig. 10. Arrangement of gauges in the two short pieces

is very similar for the two pieces and the observations in the piles. The slight temperature change beyond the first 5 days is due to the fact that the short pieces were placed in the outside and the temperature was affected by the changing ambient air temperature between April and July in Busan.

The development of strain in the PHC piece is shown in Fig. 13. Again, the strain values have not been adjusted for temperature. The values indicate a shortening of the rebars as the temperature increases followed by a recovery of the shortening during the subsequent cooling. The trend is similar to that observed in the piles. However, after the cooling is completed, the PHC piece is left with a shortening, about 50 $\mu\varepsilon$, whereas the gauges at the ground surface (#12) in the Shinho O-cell test pile indicate about 200 $\mu\varepsilon$ net elongation. A strain of about 100 $\mu\varepsilon$ corresponds to a load on the 0.28 m^2 cross section of the grouted pile equal to approximately 1,000 kN.

The development of strain in the unrestrained piece, as shown in Fig. 14, differs from that of the PHC piece, indicating the substantial effect of the restraining concrete cylinder wall on the rebar strains. In fact, it is not just that the strains are smaller, the trend during the large rise of temperature and subsequent cooling observed during the first about 15 h is reversed for the unrestrained piece. When the temperature returned to the ambient temperature, the rebar of the unrestrained piece was left with a net elongation of about 100 $\mu\varepsilon$, in contrast to the rebar in the PHC piece where the net change was a net shortening of about 50 $\mu\varepsilon$. Moreover, after the return to stable temperature at about Day 10, the PHC piece shows a gradual shortening of about 50 $\mu\varepsilon$ until Day 100, whereas the unrestrained piece shows essentially no strain change between Day 10 and Day 100.

The difference between the PHC piece ("restrained piece") and the unrestrained piece is further demonstrated in Fig. 15 showing the measured strain changes versus temperature.

After 5 months of observations, on September 29, 2006, Day 154, the two short pieces were submerged in water by means of filling the 800-mm plastic pipe around the pieces with water that then could be freely absorbed by the concrete. (Immediately before, the unrestrained piece was stripped of its plastic casting mold). Measurements were then continued for a full additional year (until 700 days after grouting), with an accidental monitoring pause between February and May 2007, days 287 and 405. Fig. 16 shows the measurements of temperature during the entire measurement period.

Immediately upon submerging, the pieces began to swell as evidenced by the progressively increasing elongation shown by the strain-gauges (Fig. 17). Significant swelling continued for about 3 months, Day 154–250, with the total swell amounting to about 150 $\mu\varepsilon$ for both pieces. Beyond Day 250, the measured strains are considered due to a combination of continued swelling

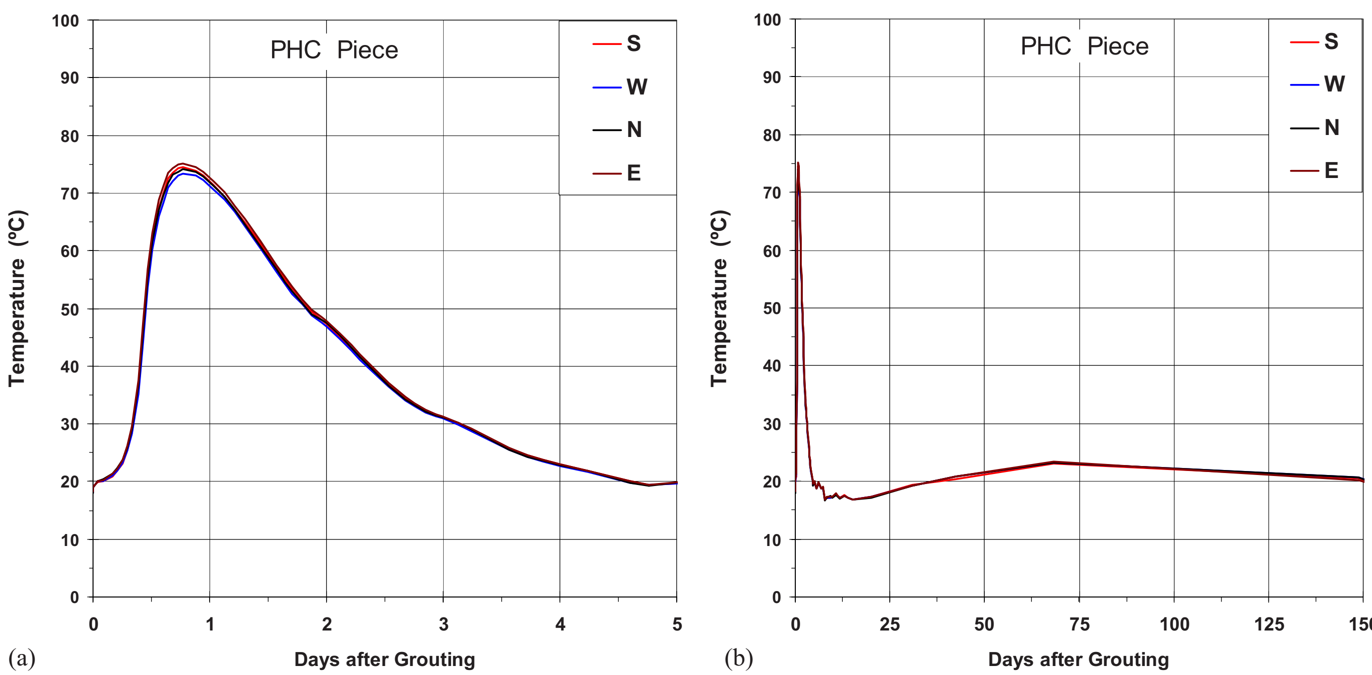


Fig. 11. Measurements of temperature in the PHC piece during first 150 days after grouting

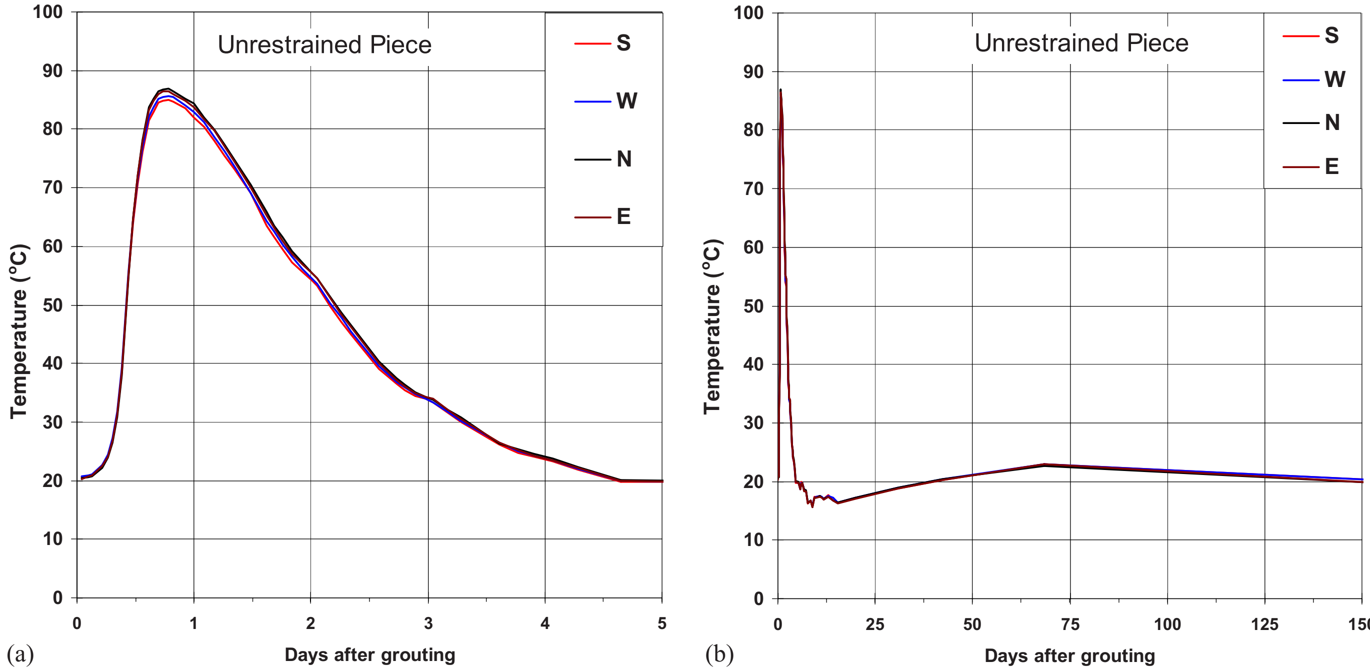


Fig. 12. Measurements of temperature in the unrestrained piece during first 150 days after grouting

and, to a minor extent, the temperature changes. The dashed lines indicate the interlude when no measurements were taken.

Discussion

A vibrating wire gauge operates on the fact that when the tension changes in a wire stretched between two supports, its natural frequency changes. The change of frequency is calibrated to the change of strain in the wire and, therefore, in the sister bar to which it is attached. The thermal coefficient of the tensioned steel wire of the vibrating wire gauge is approximately the same as the coefficient of the gauge steel housing and the steel rebar (sister bar) it is attached to. That is, when temperature changes, the wire and the housing and rebar would elongate or shorten equally and no change would occur in the wire tension, which means that the wire frequency would not change and the vibrating wire gauge itself then be unaffected by a temperature change. In reality, there

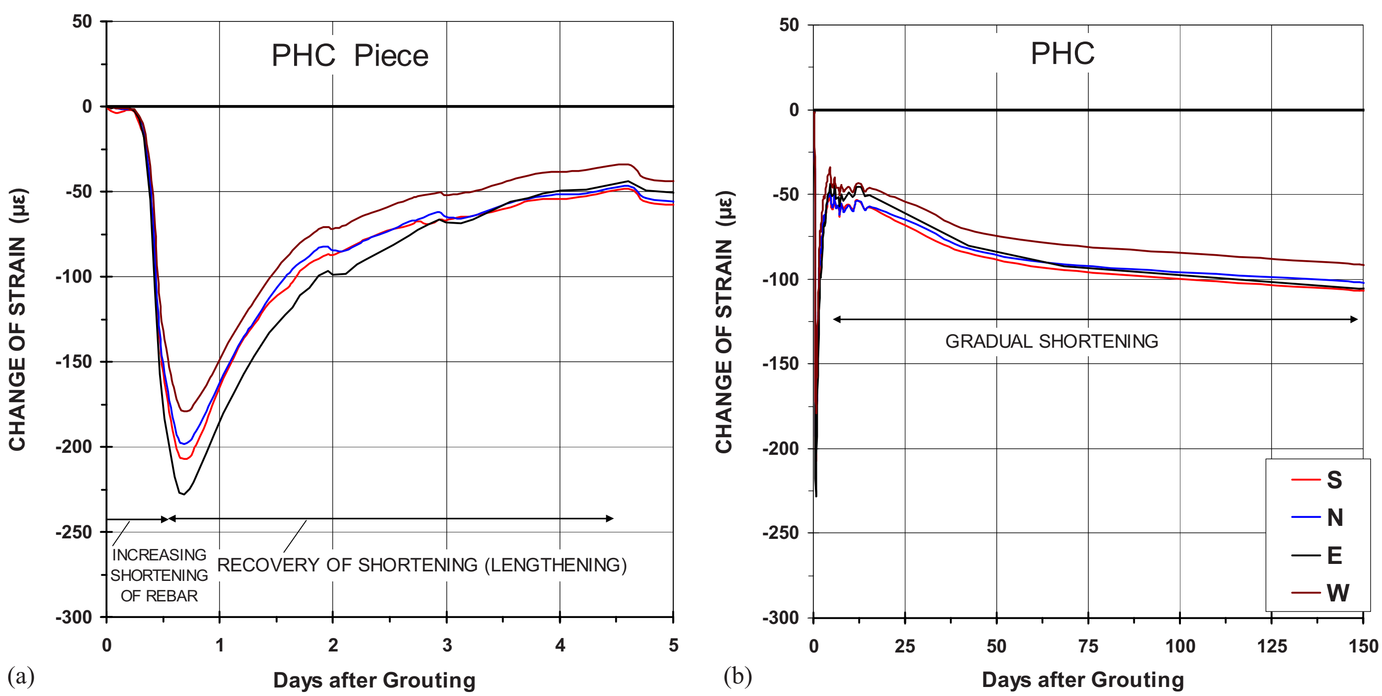


Fig. 13. Rebar strain in the PHC short piece recorded during first 5 days and the first 150 days after grouting. The strain values have not been adjusted for the difference in thermal coefficient between the rebars and the concrete and grout.

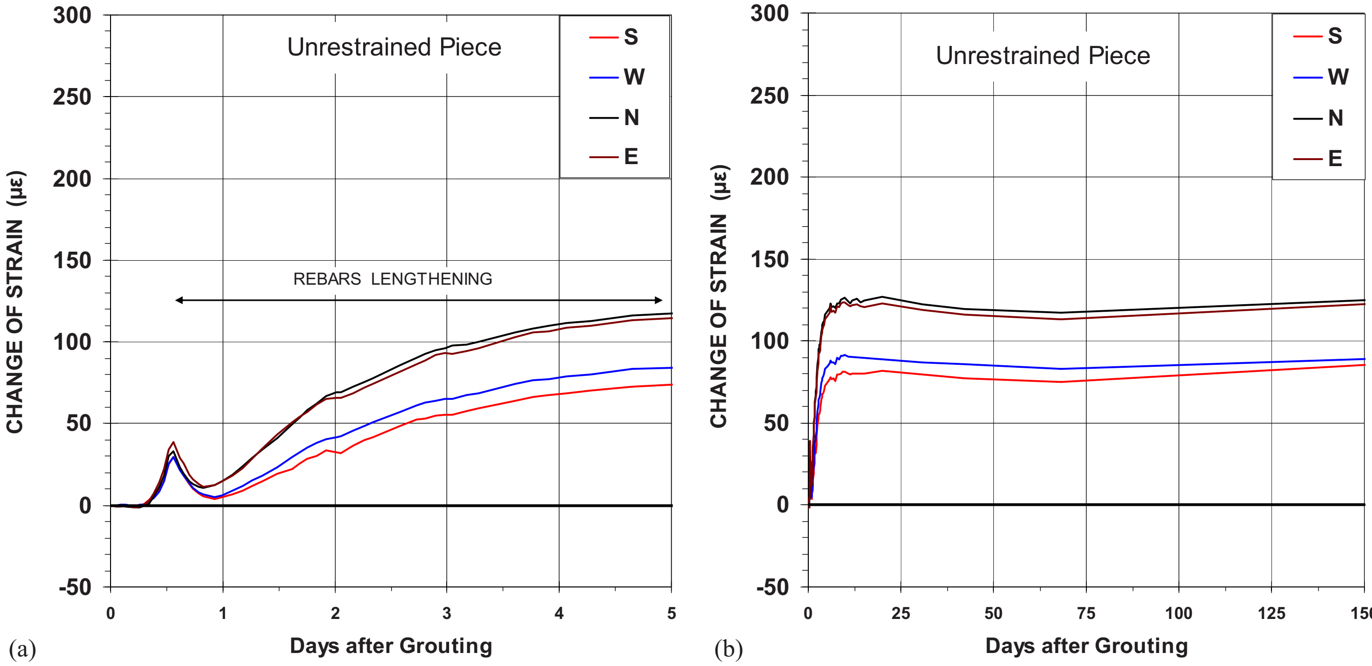


Fig. 14. Rebar strain in the unrestrained short piece recorded during first 5 days and first 150 days after grouting. The strain values have not been adjusted for difference in thermal coefficient between the rebars and the concrete and grout.

could be a small temperature dependency. The manufacturer (Sellers, personal communication, 2006) indicates an apparent decrease of strain (rebar shortening) of 0.3 $\mu\varepsilon/°C$ due to a temperature increase.

To verify the extent of change of strain due to a temperature change for the vibrating wire alone, four sister bars were placed hanging, exposed to open air, but free of all outside influence other than daily temperature variation over a couple of days. A data logger was used to record temperature and gauge readings. Two of the four gauges were of Geokon Model 4911A, the same type used in the piles, and the other two were a similar type of a Korean brand used in the short pieces. The temperature ranged from 7–16°C. Fig. 18 shows the measured variation of temperature and measured changes of unadjusted strains during the 72 h of monitoring.

The differences in strain between each recorded zero strain values to the average strain value are shown in Fig. 19. The results of the mentioned simple, narrow temperature range verification test indicate that neither gauge manufacture is affected by a temperature change. Therefore, the gauge's own temperature sensitivity is considered to be not a cause of measured strain variations for the piles and short pieces and the measured strain

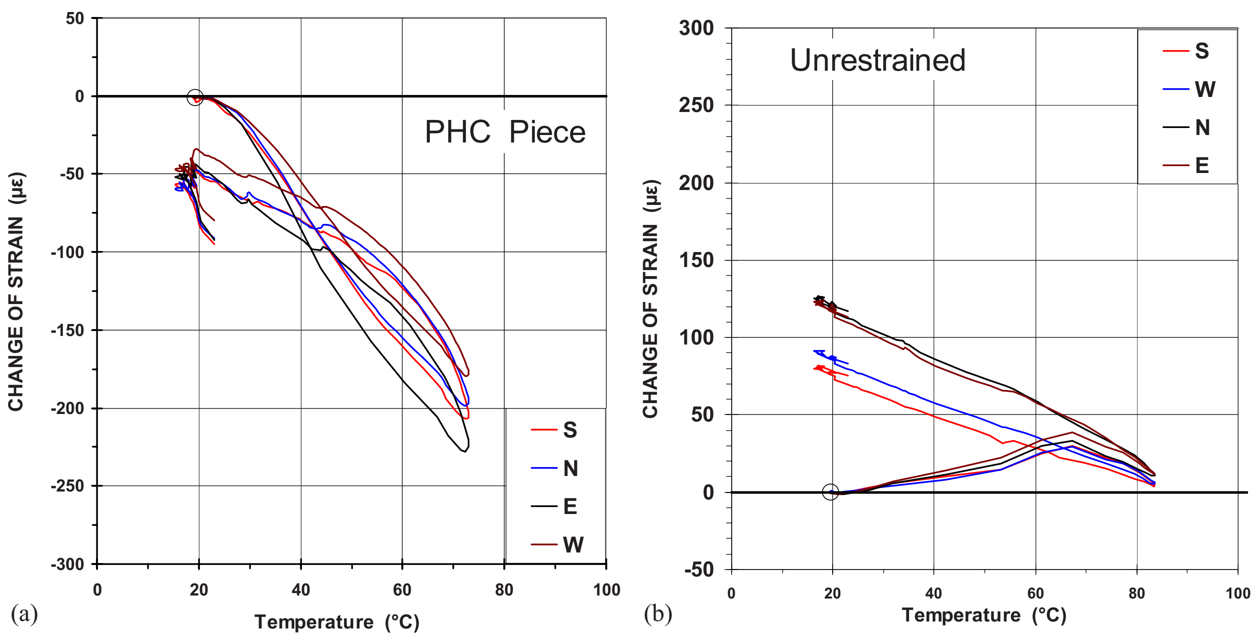


Fig. 15. Rebar strain versus temperature in the short pieces during first 150 days after grouting

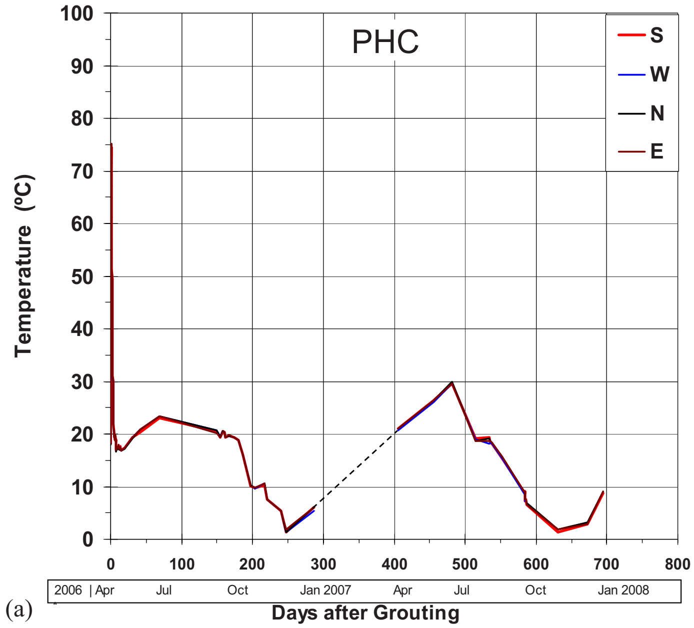


Fig. 16. Development of temperature in the short pieces during the measurement period

changes in the pile pieces are the results of hydration effects, temperature (thermal-expansion difference), and swelling, occurring more or less simultaneously. It is, of course, desirable to quantify and separate the three effects.

Shrinkage of the concrete will start very early after the casting. During the hydration, free water will chemically bond to the cement grains, which results in chemical shrinkage. Some water will also leave the mix by drying at the surface, resulting in drying shrinkage and, for the PHC pile, the concrete cylinder will rob the grout of some water. Drying and wetting will result in strain changes in addition to those caused by the temperature changes. When concrete cures under water, such as when placed below the groundwater table, as in the case of the subject full-scale piles, drying in the sense of evaporation of water to the outside does not occur. Instead, both the PHC concrete wall and the grout will absorb water from the surrounding saturated soil, which will result in swelling of the pile.

In the case of the short pile pieces, no water was absorbed during the curing, as the pieces were above the ground. Then, when the pieces were submerged, swelling occurred.

The vibrating wire sister-bar strain-gauge is usually, as in the current application, embedded in concrete or grout, whose thermal coefficient is different from that of steel. Therefore, when the temperature increases (or reduces), the differential expansion (or contraction) of the rebar and the concrete results in shear forces developing along the rebar and a strain change

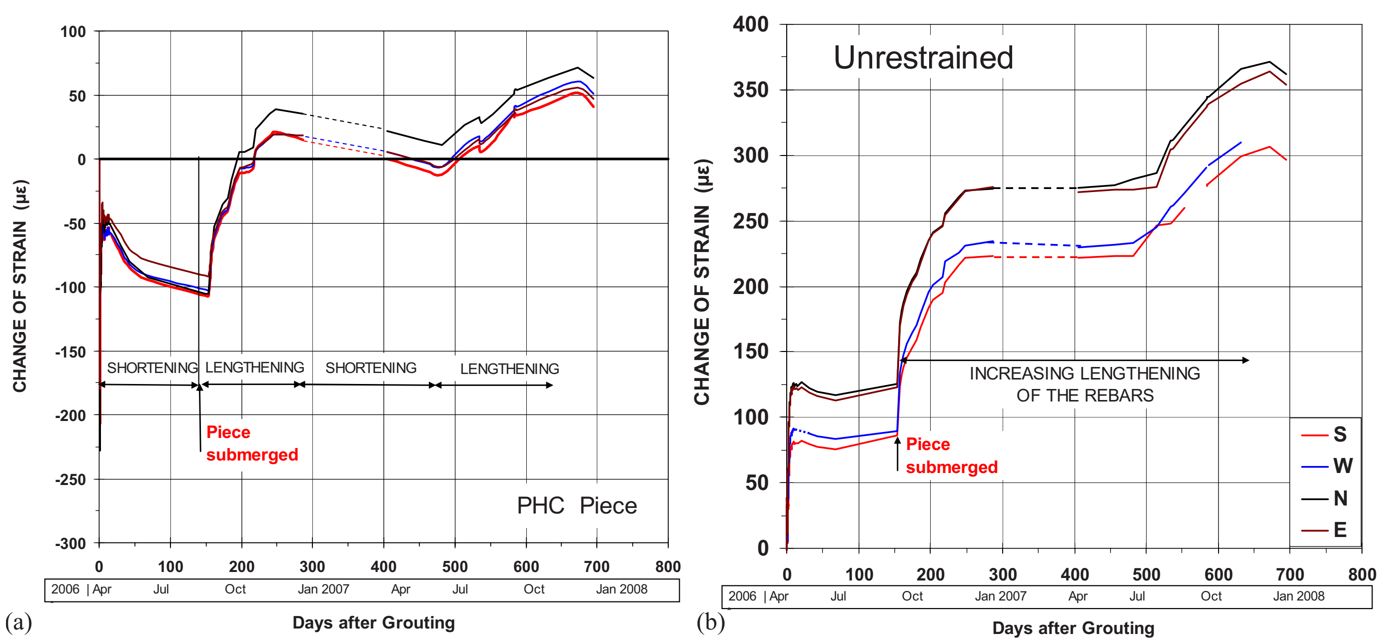


Fig. 17. Development of average rebar strain in the short pieces during the measurement period

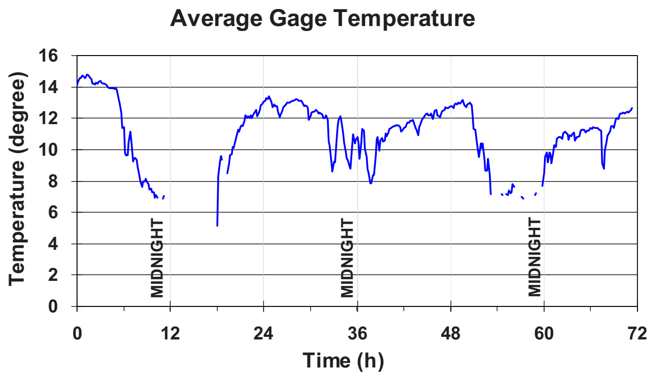


Fig. 18. Variation of temperature (°C) during the study of gauge temperature sensitivity

occurs, as expressed in Eq. (1) (Geokon Inc. 1986; Dunnicliff 1988).

$$\varepsilon_{\text{corrected}} = (R_1 - R_0)C + (T_1 - T_0)\Delta \tag{1}$$

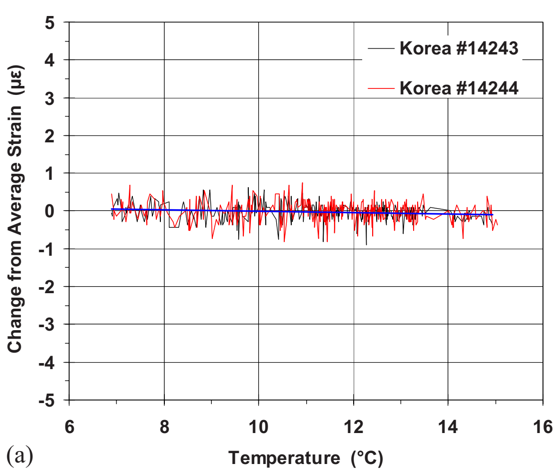
The thermal-expansion coefficient of steel is 12 $\mu\varepsilon/{}^\circ$C. However, the thermal-expansion coefficient of concrete varies widely. For cement paste, it ranges from about 18–30 $\mu\varepsilon/{}^\circ$C, when the water/cement ratio ranges from about 0.4–0.6 (Mindess et al. 1999). Powers (1968) indicated that the thermal coefficient of cement paste is 11 $\mu\varepsilon/{}^\circ$C. According to Mindess et al. (1999), the thermal-expansion coefficient of concrete aggregates varies depending on the mineralogical composition of the aggregates. For example, the coefficient of quartz is 12 $\mu\varepsilon/{}^\circ$C and for limestone, it is about a mere 6 $\mu\varepsilon/{}^\circ$C. Powers (1968) indicated that the coefficient for a particular aggregate can vary by an order of magnitude and emphasized that a plain mineralogical description cannot be used to predict the thermal coefficient, that the thermal-expansion difference coefficients between paste and aggregates "*are almost never alike*" and that the coefficients will vary with the degree of saturation of the concrete. The variations and ranges, notwithstanding, is commonly assumed that the thermal-expansion coefficient of concrete is 10 $\mu\varepsilon/{}^\circ$C and that, therefore, the thermal-expansion difference coefficient between steel and concrete is 2 $\mu\varepsilon/{}^\circ$C. The manufacturer's recommended value is 2.2 $\mu\varepsilon/{}^\circ$C (Geokon Inc. 1986), i.e., concrete expands and contracts at a rate slightly smaller than that of the rebar. The coefficient of a cement-paste grout could be both larger and smaller than that of steel, i.e., the thermal-expansion difference coefficient could be negative. Most likely, the concrete wall of the PHC pile, making up half the volume of the PHC short piece, has a somewhat disparate thermal-expansion coefficient as opposed to the cement-paste grout making up the center of the PHC piece and of the unrestrained piece.

For the subject PHC pile, when temperature increases, an encased rebar will want to elongate more than the concrete, and, as the concrete will impede some of the elongation, a strain gauge attached to the rebar will indicate a compression. Similarly, at the same time, the reinforcement will tend to "pull" at the concrete causing tension strain in the concrete.

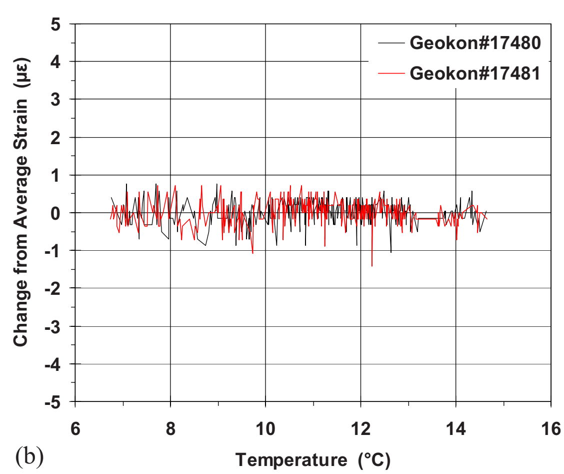
To obtain strain readings compensated for a temperature change, the strain value of the rebar-mounted gauge needs to be corrected, according to the relation shown in Eq. (1). The first term in Eq. (1) becomes negative and, as the thermal-expansion difference coefficient is positive for a rebar-mounted gauge, the second term is positive and equal to the first term—the net result is zero strain. However, the records from both the PHC piles and the PHC piece show increasing strain as an effect of the rise of temperature due to the start of the hydration of the fresh concrete. Clearly, Eq. (1) does not apply to the hydration period of the grouted PHC pile.

For fresh concrete, Cusson and Hoogeveen (2007) indicated that stresses start to develop in a fresh concrete at time zero, which they define as the time when temperature starts to rise. They indicated that the thermal-expansion coefficient of fresh concrete at time zero is slightly smaller than at the peak temperature and is approximately constant thereafter. For the short-piece test, time zero occurred at about 4 h after casting. At this time, the strain-gauges would start to register compression strain in the bars caused by the unequal thermal-expansion coefficients for the steel and the grout.

After the dissipation of the temperature rise and when the hydration effect is over, which appears to have occurred about 10 days after the start, a recorded strain change can be assumed to be caused by temperature change alone. If so, the recorded strain values should calibrate the thermal-expansion difference coefficient, i.e., a suitable coefficient should eliminate all temperature-induced strain changes in about 10 days and until the pieces were submerged at 154 days. As indicated in Figs. 11 and 12, the temperature change during this time was rather small, which is only



(a)



(b)

Fig. 19. Recorded variation of zero strain reading during the 72 h observation period

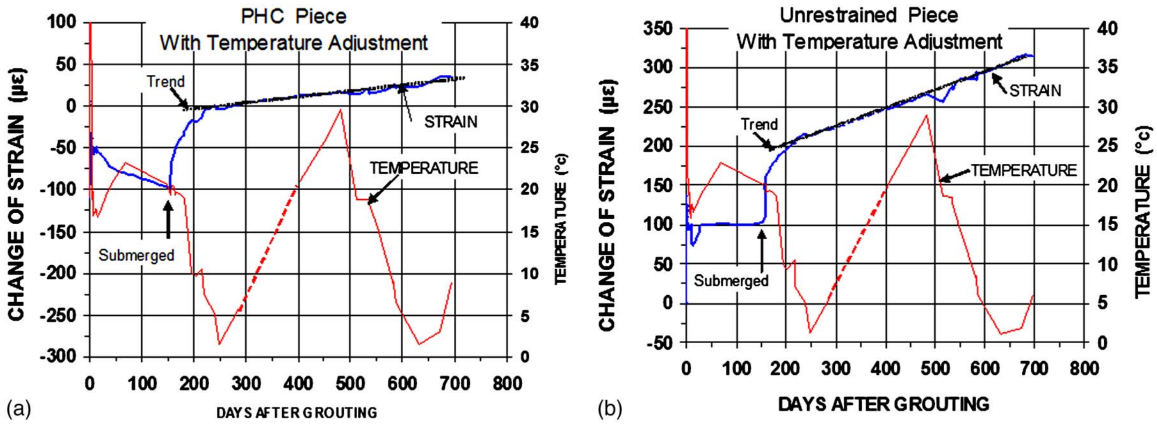


Fig. 20. Average rebar strains when the effect of temperature is removed by means of applying a temperature-expansion difference coefficients of 1.7 and 1.8 for the PHC and the unrestrained short pieces, respectively

3°C. Figs. 13 and 14 show that the strain change was a slight shortening of the PHC rebar, whereas no appreciable change occurred for the rebars in the unrestrained piece. Because of the stable temperature, no conclusion as to the thermal coefficients can be drawn from the results of the laboratory test before Day 154.

The strain changes during the period immediately after submerging the pieces are due to both the swelling and the effect of temperature change. It is assumed that if swelling was acting alone, the strain-time curve would be reasonably smooth and show a steady trend. As it was established that a change of temperature has minimal or no effect on the strain-gauge readings themselves, a suitable temperature-expansion coefficient can be deduced to correct the strain values for temperature, that is, eliminate the change of strains in the rebars due to the change of temperature. Figs. 20 and 21 show the strain-time curve for the PHC and the unrestrained pieces, respectively, when applying thermal-expansion difference coefficients of 1.7 and 1.8, respectively.

The strains remaining after the temperature adjustments are considered those caused by the swelling alone. After the initial rapid swelling and after about 2 months, the trend is linear with time and continuous for the duration of the measurements. The strains during the first 100 days after submerging are essentially equal for the two pieces. Thereafter, however, the rate of increase in the PHC piece is about 2 $\mu\varepsilon$/month (10 $\mu\varepsilon$/80 days) while it is three times larger for the unrestrained piece. The difference is thought to be due to that the concrete wall is less able to swell than the concrete grout. However, as it is also more pervious than the grout, the wall can absorb water faster than the grout, which compensates for its smaller swelling ability and, coincidentally, makes the initial swelling rate about the same for the two pieces. As the water reaches the grout, the PHC wall restrains the swelling caused by the water absorbed by the grout, which does not happen for the unrestrained piece. While a difference of 2 $\mu\varepsilon$ between two values measured some time apart could be due to gauge drift, the observed changes of strain due to swelling over the 2-year measurement period accumulated to about 50 and 150 $\mu\varepsilon$, respectively, which is much too large to be considered due to any gauge drift.

The results of the measurements of strain and temperature in the Shinho PHC pile are shown in Fig. 22. For clarity, only the data of three gauge levels are shown: Level 12, which is at the ground surface, Level 11, which is at a depth of 4 m, and Level 2,

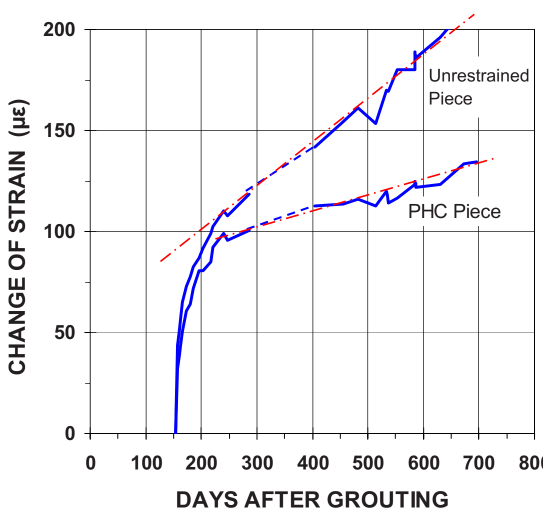


Fig. 21. Strain due to swelling alone of the short pieces after 154 days

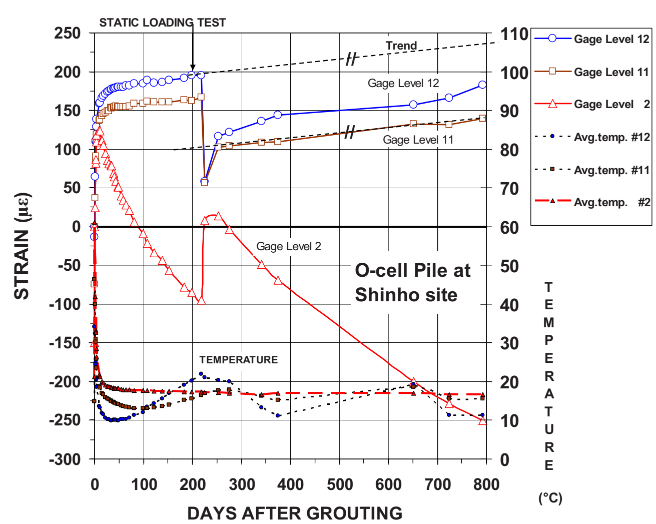


Fig. 22. Temperatures and temperature-adjusted strains in the Shinho pile

which is a depth of 49 m. Gauge Level 12 is affected by temperature variation and so is Level 11 but to a lesser degree. No variation of temperature occurs at Level 2. The pile is equipped with an O-cell at the toe and an O-cell static loading test was performed at Day 218 followed by a head-down static loading test. The strain data are adjusted by the temperature-expansion difference coefficient of 1.7 established from the short-piece test. The elongation trend after the end of the static loading test (about Day 250) for the uppermost gauge levels (#12 and #11) is about 2 $\mu\varepsilon$/month, that is, the same as for the PHC short piece.

The significant strain reduction at gauge Level 2 is due to the ongoing build-up of residual load in the pile with time after the static loading tests. The build-up is partially due to the reconsolidation of the soil and, possibly, also to a part to the soil restraining the pile from swelling. Analysis of the measurements in the full-scale piles are not a subject for this paper, however, but will be discussed in a separate paper. It is noted, however, that for the test piles and for any concrete pile or grouted pile, the strains caused by the hydration process and temperature effects will have significant influence on the strains in the pile. Neglecting this influence will adversely affect the interpretation of the gauge readings during a static loading test or long-term observation test.

Conclusions

The observation in the full-scale piles showed that the temperature increased due to the curing of the concrete grout to 60 and 85°C in the Shinho and Myeongji PHC piles, respectively, and to 60°C in the Golden Ears bored pile. The peak temperature was reached after about 20 h in the two PHC piles, whereas it took 2 days for the maximum temperature to be reached in the bored pile. The subsequent cooling to ground temperature took about 5 days in the PHC piles and more than 30 days in the bored pile.

Simultaneously with the hydration of the grout, the strains recorded by the sister-bar gauges in the piles showed that the hydration imposed a shortening of the rebars during the temperature rise followed by a recovery of the shortening and a net elongation. For the two PHC piles, the shortening was about twice as large for the Myeongji pile which had shown the largest temperature increase. The net elongation ranged from about 100–300 $\mu\varepsilon$. The bored pile showed no similar net elongation. After 150 days for the PHC piles and 30 days for the bored pile, the net strains in the PHC piles amounted to an elongation of about 200–300 $\mu\varepsilon$ for the uppermost gauges in the Shinho and Myeongji PHC piles, respectively, while the Golden Ears bored pile showed a net shortening of 200–300 $\mu\varepsilon$.

The recorded shortening during the temperature rise in the PHC piles is due to the rebar wanting to elongate more than the grout and is restrained from doing so, which results in compression strain in the rebar. A gauge embedded in the concrete would have shown a corresponding strain increase (tension). With temperature correction, both gauges would show no strain change due to a temperature change and equal response to an outside load.

The reason for the difference between the PHC piles and the PHC short pieces during the cooling following the temperature increase is considered due to that, in the field, the cylinder wall of the PHC pile can absorb water and will swell, which introduces an elongation of the pile as recorded by the strain gauges.

The reconsolidation of the soil after the driving of the PHC pile and the sample bored pile imposed residual load in the pile that was larger and deeper down the pile where the recording gauge is located. The resulting compression (shortening) of the pile was partially offset by the swelling of the PHC pile and was fully offset by the swelling of the bored pile.

The temperature measurements in the short pieces showed that the cylinder wall of the PHC pile had a cooling effect with a slightly lower maximum temperature, about 75°C, in the recorded PHC piece than the unrestrained piece, about 85°C.

The imposed change of strain in the PHC piece is similar to that of the full-scale piles in showing an initial shortening followed by recovery of the shortening. No outside source of water was available that could have caused a following elongation.

The submerging of the short pieces resulted in significant swelling due to the water being absorbed by the short pieces. The records are affected by the simultaneously occurring temperature change. However, as the trend of the swelling can be assumed to be uniform, the temperature influence was eliminated by fitting a temperature-expansion difference coefficient to the strain values of the PHC and the unstrained short pieces, respectively. The so fitted coefficients were of 1.7 and 1.8, which are close to the manufacturer's recommended temperature-expansion difference coefficient value of 2.2. The swelling influence was at first very similar for the two short pieces but, beyond the first 100 days after submerging, a linear trend developed that amounted to 2 $\mu\varepsilon$/month for the PHC piece and 6 $\mu\varepsilon$/month for the unrestrained piece. The trend continued for the duration of the measurements.

The Shinho full-scale PHC pile showed the same linear 2 $\mu\varepsilon$/month swelling trend as the PHC unrestrained piece.

The short-piece measurements show that the observations of about 200–300 $\mu\varepsilon$ elongation in the uppermost strain-gauge levels of the full-scale Shinho and Myeongji PHC test piles during the first 10 days after grouting is due to the effect of the curing of the grout and the succeeding measured elongation is due to swelling of the piles as they absorb water from the soil. It appears reasonable to assume that the strain development due to the curing and the swelling of the gauges deep down is the same as the uppermost gauges. The strains measured for the uppermost gauges can therefore be used to adjust the strains measured for the gauges deep in the soil to determine the amount of residual load that had been built-up in the test piles at the start of the static loading test.

It is noted that for the test piles and for any concrete pile or grouted pile, the strains caused by the hydration process and temperature effects will have significant influence on the evaluation of load in the test pile. Neglecting this influence will adversely affect the interpretation of the gauge readings during a static loading test or long-term observation test.

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Notation

The following symbols are used in this paper:

C = gauge calibration factor ($\mu\varepsilon\ 10^3/f^2$);
R_0 = initial reading (digits=$f^2 10^{-3}$; frequency squared divided by 1,000);
R_1 = current reading (digits=$f^2 10^{-3}$; frequency squared divided by 1,000);
T_0 = initial temperature (°C);
T_1 = current temperature (°C);
α_C = thermal-expansion coefficient of concrete ($\mu\varepsilon/°C$);
α_S = thermal-expansion coefficient of steel ($\mu\varepsilon/°C$);
Δ = thermal-expansion difference coefficient $=\alpha_S$-α_C ($\mu\varepsilon/°C$); and
$\varepsilon_{\text{corrected}}$ = strain corrected for temperature change ($\mu\varepsilon$).

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