SOILS AND FOUNDATIONS Vol. 43, No. 6, 155-171, Dec. 2003
Japanese Geotechnical Society

PERFORMANCE OF A PRELOADED-PRESTRESSED GEOGRID-REINFORCED SOIL PIER FOR A RAILWAY BRIDGE

TARO UCHIMURA[i], MASARU TATEYAMA[ii], IKUO TANAKA[iii] and FUMIO TATSUOKA[iv]

ABSTRACT

A new construction method, called "the preloaded and prestressed reinforced soil method", proposed in this paper, aims at making reinforced backfill structures very stiff and stable. To make the deformation of a reinforced backfill nearly elastic, sufficiently large preload is first applied by introducing tension into metallic tie rods that penetrate the reinforced backfill and are connected to top and bottom reaction blocks. High tensile force in the tie rods functions as prestress, increasing the confining pressure in the backfill and thus keeping the stiffness and shear strength of the backfill soil sufficiently high. In 1996, in northern Kyushu, Japan, a prototype pier of preloaded and prestressed geogrid reinforced backfill was constructed for the first time to support a pair of simple beam girders for a temporary railway bridge. An abutment of geogrid-reinforced soil retaining wall, which was neither preloaded nor prestressed, was also constructed for the same bridge by otherwise the same construction method. The behaviours of the pier and the abutment were measured during the construction and the service period of about four and a half years and subsequently full-scale loading tests were performed. It is shown that the geogrid-reinforced backfill pier became substantially stiffer against static and dynamic load by having been preloaded and being prestressed when compared to the geogrid-reinforced backfill abutment.

Key words: case history, cyclic loading, deformation, (long-term performance), preloading, (prestressing), reinforced soil (**IGC**: E12/K14)

INTRODUCTION

Tatsuoka et al. (1997a and b) proposed a new construction method, called the preloaded and prestressed (PLPS) reinforced soil method. This method aims at making reinforced backfill very stiff and very stable against vertical load applied at the crest of the backfill, as well as seismic load by preloading and prestressing the backfill vertically. It was expected that reinforced soil structures constructed by this method could support massive important structures without exhibiting intolerable deformations. Adams (1997) and Ketchart and Wu (1997) proposed a similar construction method. However, the preload is fully removed after the backfill is preloaded in their method, not taking advantage of prestressing while the structure is in service.

Several important advantages of the PLPS reinforced soil method, as described in this paper, were confirmed from high performance of both field full-scale models of PLPS reinforced soil retaining wall (Uchimura et al., 1996, 2001) and small-scale models of reinforced soil structure constructed in the laboratory (Shinoda et al., 1999, 2003). Uchimura et al. (1996) performed triaxial compression tests on 30 cm-dia. specimens of the same type of gravel as used as the backfill material for the prototype structure described in this paper. In these model tests and triaxial tests, creep loading and stress relaxation tests were performed during primary loading and under preloaded and prestressed conditions.

The first prototype PLPS geogrid-reinforced backfill structure was constructed in northern Kyushu, Japan, in July 1996 as a pier supporting a pair of simple beam girders for a temporary railway bridge. The bridge was opened to service in mid August 1997 and was used until the end of March 2001. An abutment of geogrid-reinforced backfill was also constructed for the bridge, which was, however, neither preloaded nor prestressed. A comparison of the behaviours of the PLPS pier and the abutment showed that the preloading and prestressing procedure is very efficient for restraining creep deformation of the backfill under long-term static loading conditions as well as transient and residual deformation caused by long-term traffic load. Part of this case history at intermediate stages has been reported by Uchimura (1998) and Shinoda et al. (2003). In this paper, a full description of the case history until the end of service of the bridge is

i) Assistant Professor, University of Tokyo (uchimura@geot.t.u-tokyo.ac.jp).
ii) Manager, Railway Technical Research Institute.
iii) Manager, Construction Division, Kyushu Railway Company.
iv) Professor, University of Tokyo.
Manuscript was received for review on October 7, 2002.
Written discussions on this paper should be submitted before July 1, 2004 to the Japanese Geotechnical Society, Sugayama Bldg. 4F, Kanda Awaji-cho 2-23, Chiyoda-ku, Tokyo 101-0063, Japan. Upon request the closing date may be extended one month.

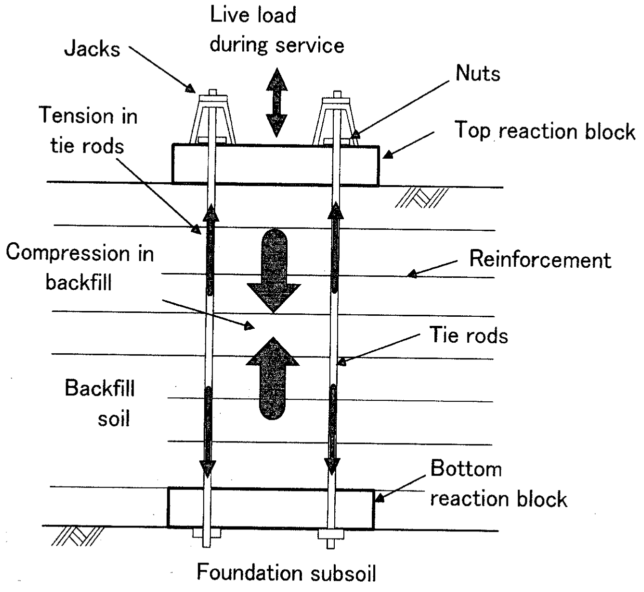

Fig. 1. Schematic diagram of preloaded and prestressed reinforced backfill

given. After the end of service of this bridge, full-scale loading tests of the pier and abutment were performed to confirm the high performance of the PLPS reinforced soil pier observed during service. A part of the results from the full-scale loading tests is also reported in this paper, while a full-description will be given in the companion paper (Uchimura et al., 2004).

OUTLINE OF PLPS REINFORCED SOIL METHOD

The typical construction procedures of the PLPS reinforced soil method are as follows (Tatsuoka et al., 1997a and b) (Fig. 1):

1. If necessary, the foundation sub-soil is improved, for example, by cement-mixing-in-place. Relatively expensive pile foundations are not used because of an inherently high flexibility of reinforced backfill structures.
2. A rigid bottom reaction block made of reinforced concrete (RC), or its equivalent, is constructed on the foundation sub-soil.
3. Four steel tie rods are installed vertically through the backfill with their bottom ends anchored into the bottom reaction block.
4. The backfill is constructed by using, for example, geogrid reinforcement while holding the tie rods inside. The use of well-graded gravel is preferred for the backfill. A high degree of compaction of the backfill is essential.
5. A top RC reaction block is constructed on the top of the completed backfill.
6. A set of hydraulic jacks is set at the top ends of the tie rods, supported by the top reaction block.
7. The backfill is vertically preloaded by using the jacks. To develop as large as possible compression of the backfill at this stage, a sufficiently high preload is kept constant for a long period or a cyclic load with a number of cycles is applied.
8. The preload is partially released from the preload level to a prescribed non-zero prestress level.
9. The top ends of the tie rods are fixed to the top reaction block by using nuts before removing the jacks. A more sophisticated device, called the ratchet connection system (Shinoda et al., 2003), could be used in place of the nuts to maintain the prestress for a longer duration.
10. The vertical stress remaining in the backfill is in equilibrium with the tie rod tension, which works as prestress. It is necessary to keep the prestress as close as possible to the initial value while the structure is in service.

The basic mechanisms of this method could be summarized as follows (Tatsuoka et al., 1997a and b):

1. Much higher preload can be applied to reinforced backfill without damage than in the case of unreinforced backfill.
2. A process of preloading and its subsequent partial unloading can make the backfill very stiff and nearly elastic against external vertical load subsequently applied on the top of the structure.
   a) The amount of unloading from the preload level should be larger than the maximum design load that would be applied during service; otherwise, the compressive stress activated in the backfill during service may exceed the maximum stress during preloading, which may result in intolerable large irreversible deformations of the backfill (Shinoda et al., 1999, 2003).
   b) On the other hand, the prestress should not be very low so as to avoid excessive swelling and associated significant softening of the backfill (Tatsuoka et al., 1997c; Shinoda et al., 1999, 2003). It is essential to keep a sufficiently high prestress to ensure a high structural integrity of reinforced backfill subjected to monotonic and cyclic vertical and lateral load (Shinoda et al., 1999, 2003; Uchimura et al., 2001).
3. Vertical load working on the top of the backfill is always in equilibrium with the sum of external vertical load applied on the top reaction block and tie rod tension. When external vertical compressive load is applied, the tie rod tension decreases associated with vertical compression of the backfill. This behaviour results in a reduction of the vertical load that is applied to the backfill, reducing the compression of the backfill.
4. A large part of the tensile strains in the reinforcement layers that develop by preloading remaining after the preload is decreased to the prestress level. Therefore, the reinforcement confines the backfill deformation more efficiently when compared with the case not using the PLPS procedure.
5. When the backfill is preloaded to a sufficiently high level for a long period, relatively large creep deformation takes place in the backfill. Then, the rate of relaxation of tie rod tension under the prestressed

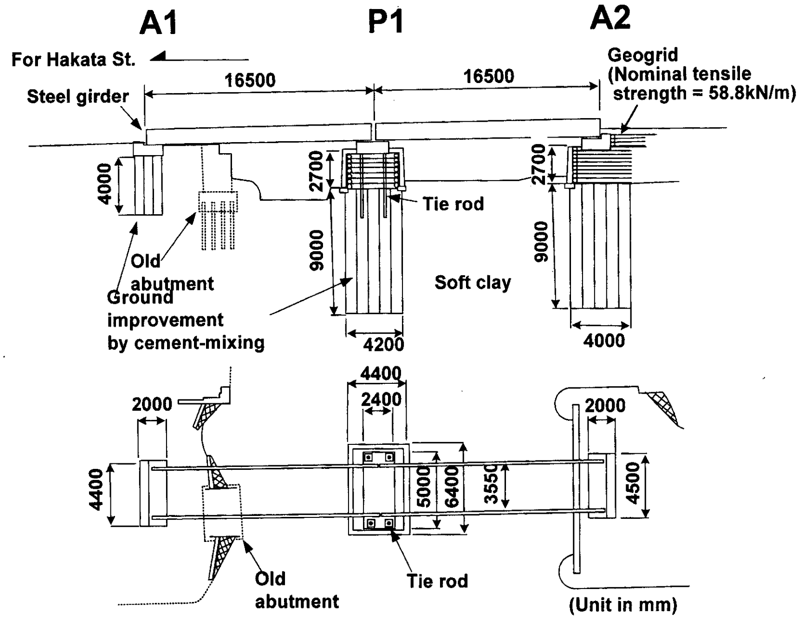

Fig. 2. Maidashi bridge with PLPS reinforced soil pier

condition becomes very low.

6. The backfill may deform both in a bending mode and in a simple shear mode during a seismic event. If the backfill is very well compacted, large dilatation of the backfill develops associated with large simple shear deformation. Such bending and dilatation modes of deformation are effectively restrained by a considerable increase in the tie rod tension, preventing a substantial decrease in the pressure level in the backfill (Uchimura et al., 2001). Therefore, a high seismic stability can be expected.

PROTOTYPE BRIDGE PIER OF PLPS GEOGRID-REINFORCED BACKFILL

Figure 2 shows Maidashi Bridge, which was constructed in July 1996 in Fukuoka City, Japan. The pier, denoted as P1 in Fig. 2, is the first prototype bridge pier consisting of a PLPS geogrid-reinforced backfill, which was constructed to support two 16.5 m-long simple beam steel girders for a single railway track. The cross-section of the pier was 6.4 m × 4.4 m and the height of the backfill was 2.4 m (Fig. 3(a)). The design dead load due to the girder weight is 196 kN and the design live load due to the weight of two locomotives, including impact load, is 1,280 kN.

First, an approximately 9 m-thick very soft clay deposit that was to support the pier and the abutment was improved by cement-mixing-in-place, forming 9 m-long and 0.8 m-in-diameter cement-mixed soil columns at each place (*see* Figs. 2 and 3(a) and (b)). In addition, the surface zone of the clay deposit within the whole cross-section of the pier, having a cross-section of 4.4 m × 2.4 m and a thickness of 1 m, was fully improved by cement-mixing-in-place to form a bottom reaction block for tie rods.

Four steel tie rods, which were originally produced for prestressing concrete beams, were prepared. The nominal yield tensile force of each tie rod is 1,034 kN. Their lower ends were anchored into the cement-mixed columns in the sub-soil. A pair of 22 mm-thick steel plates was placed on the bottom reaction block (Photo. 1). Each pair of tie rods was fixed to the respective steel plate by using nuts in order to ensure rigid anchoring into the sub-soil. The top ends of the tie rods were extended upwards by using coupling nuts on the top RC block. The tie rods were covered with PVC pipes to avoid friction with the backfill (Photo. 3).

The backfill was constructed with the help of gravel-filled bags that were stacked at the shoulder of each gravel layer along the periphery of the structure (Photo. 2). The bags were wrapped around with the respective geogrid reinforcement sheet. A well-graded gravel of crushed sandstone (D_{max} = 30 mm, D_{50} = 8–11 mm and U_c = 4.0–4.3, *see* Fig. 4) was used as the backfill material. A hand-operated 30 kg-vibration compactor and a hand-operated 60 kg-tamper were used to compact the backfill (Photo. 3). Larger machines could not be used because of severe space restraint at the site. The dry density of the backfill measured when demolished ranged from 1.91 to 2.17 g/cm^3. This density value means 80–91% of the maximum dry density (2.38 g/cm^3) at the optimum water content (3.7%) evaluated for a compaction energy of 3 × 10^6 mN/m^3. The angle of internal friction, $\phi = \arcsin\{(\sigma_1 - \sigma_3)/(\sigma_1 + \sigma_3)\}_{max}$, was 60° at a confining pressure of 49 kPa evaluated by drained triaxial compression tests on specimens (23 cm × 23 cm × 57 cm-high) having an initial dry density of 1.95 g/cm^3 (Uchimura et al., 1998).

The reinforcement used was a geogrid made of polyvinyl alcohol, coated with polyvinyl chloride (PVC). According to the results from tensile tests at a strain rate of 5%/min using a 5 cm-wide and 40 cm-long specimens

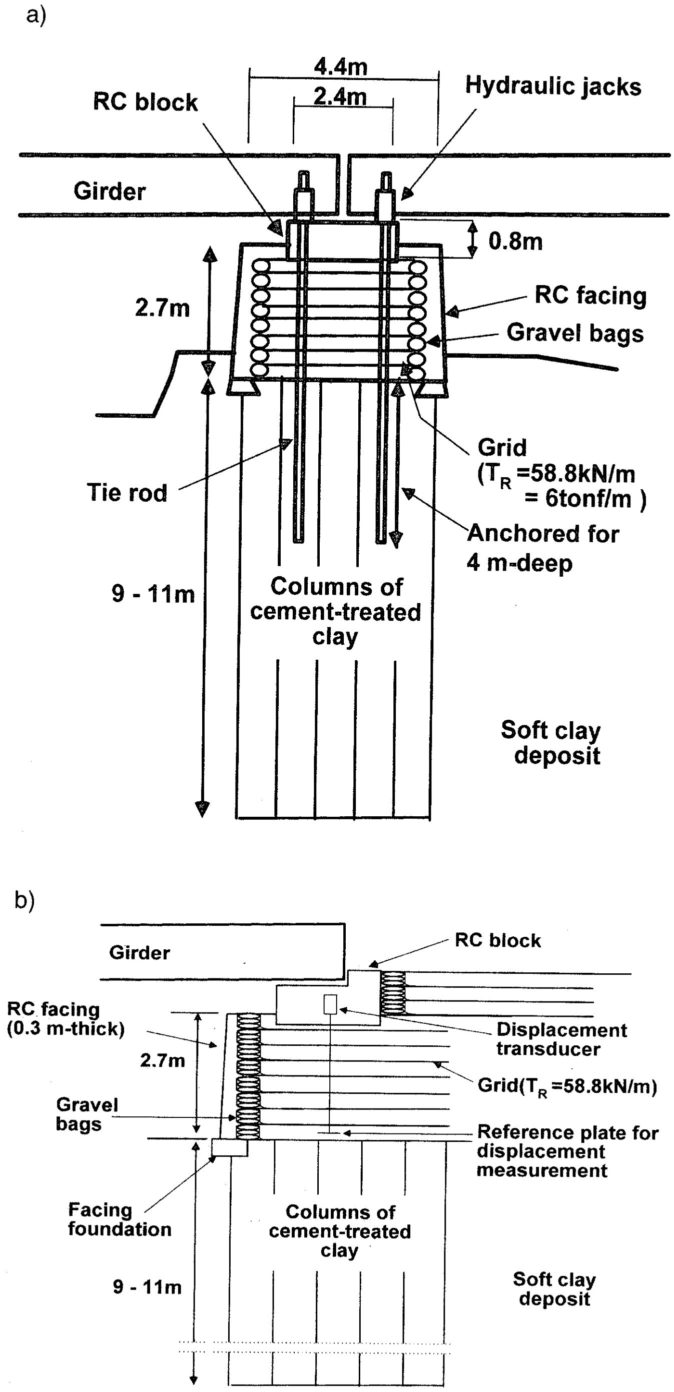

Fig. 3. a) PLPS reinforced soil pier and b) reinforced soil retaining wall abutment

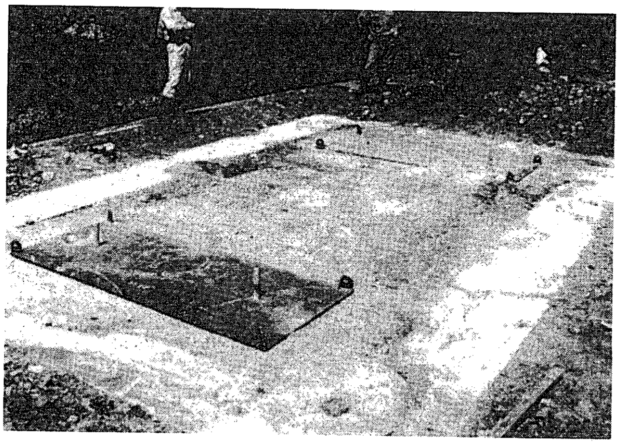

Photo. 1. Bottom of the backfill (pier)

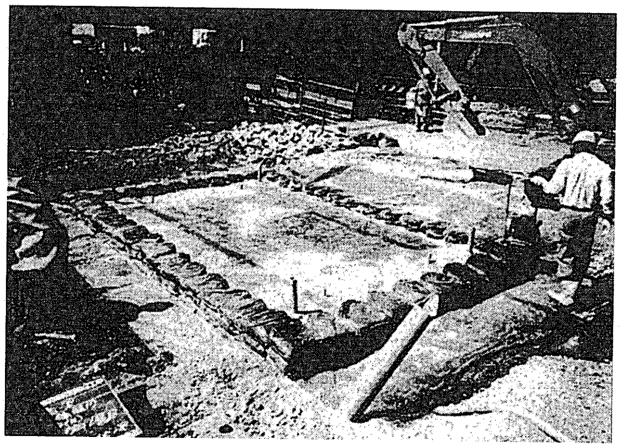

Photo. 2. Construction of the first gravel layer (pier)

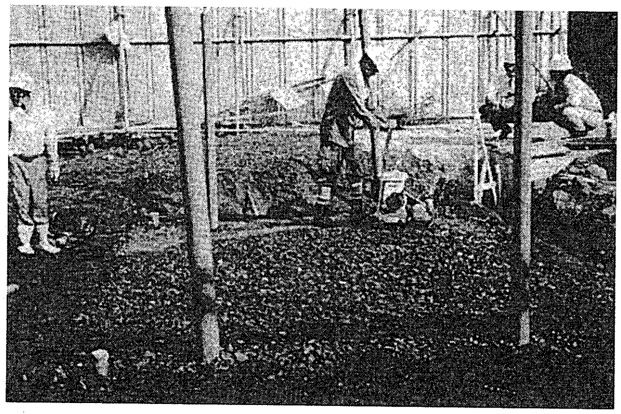

Photo. 3. Compaction of the backfill material (pier)

performed by the provider, the nominal rupture strength is 58.8 kN/m and the nominal stiffness is 1,050 kN/m at strains less than 1.0%. The arrangement of reinforcement layers was determined by the following very conservative method. The pier was treated as a geogrid-reinforced soil retaining wall (GRS-RW) having a full-height rigid facing at one side of the structure while having the same height as the actual pier. It was assumed that the structure behaves under plane strain conditions despite the rectangular prismatic shape of the actual pier. The vertical spacing of the reinforcement thus determined was equal to 30 cm. As the pier had two pairs of wall faces in two orthogonal directions, the two elevation sections, respectively having one pair of wall face, were designed independently. By overlapping the two elevation section, the actual average vertical spacing of reinforcement layers became 15 cm.

The construction of the backfill by a team of five workers took five days. The preloading work started ten days after casting-in-place the top reaction RC block (5 m-long, 2.4 m-wide and 0.8 m-thick). Photograph 4

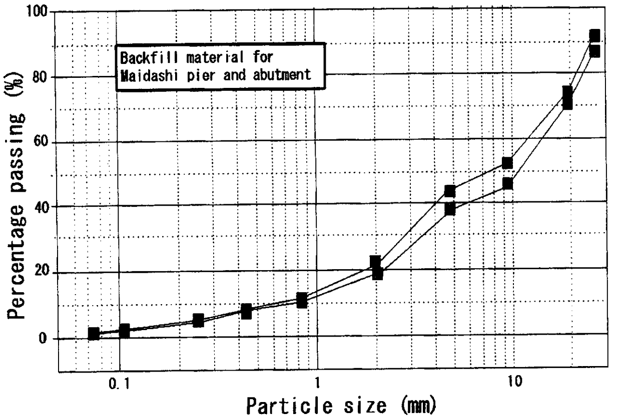

Fig. 4. Grain size distributions of the backfill material for the pier and abutment

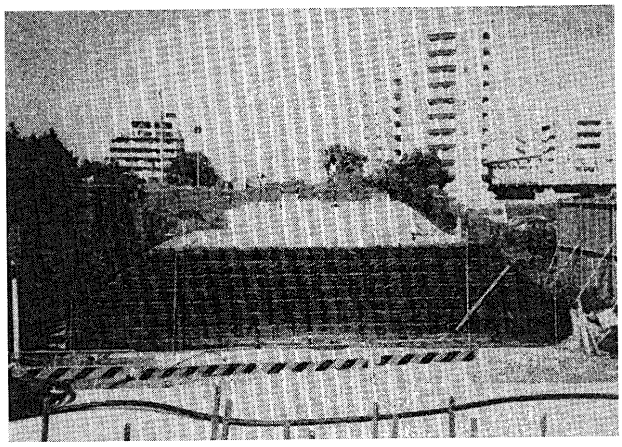

Photo. 6. Abutment before casting-in-place facing

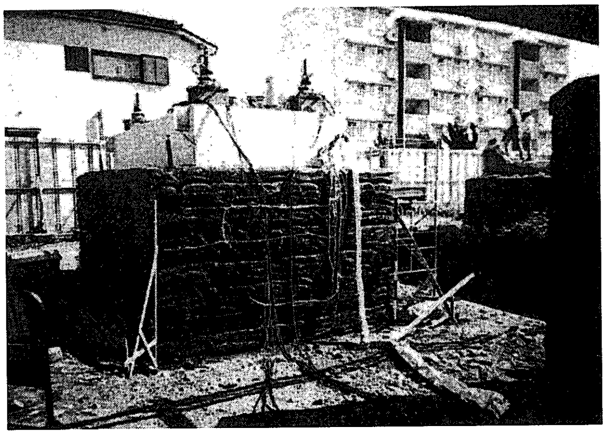

Photo. 4. Preloading (pier)

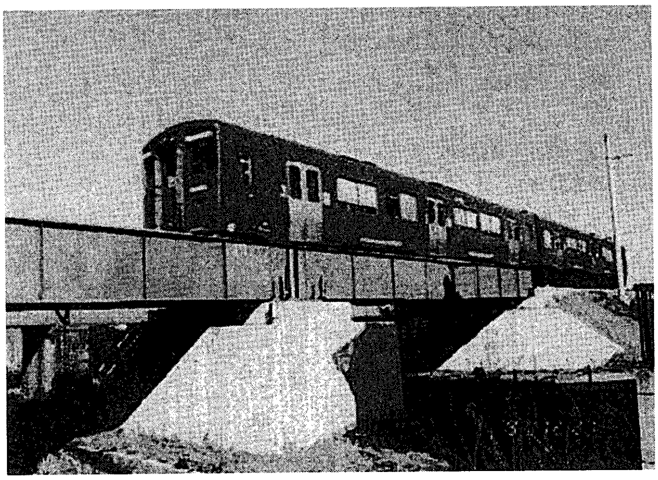

Photo. 5. Completed bridge with a PLPS pier and a GRS abutment (27 December, 1997)

shows the structure during the preloading work. After the preloading work, 30 cm-thick full-height rigid facings of lightly steel-reinforced concrete were cast-in-place on the four wall faces (Photo. 5). The total construction period by this stage was about 1.5 months.

The abutment, denoted as A2 in Fig. 2, is a conventional type geogrid-reinforced soil retaining wall (GRS-RW), which was constructed as one of the two abutments of the bridge. Figure 3(b) shows the details of the abutment. Photographs 6 and 5 show the abutment before and after placing a full-height rigid facing, respectively. The abutment was constructed by the same method as the pier, except that it had only one wall face. The range of dry density of the gravel backfill measured when demolished was 2.08–2.19 g/cm^3, which is slightly higher than that of the backfill of the pier. The backfill was reinforced with the same type of geogrid as used for the pier at a vertical spacing of 30 cm. Both sides of the backfill consisted of exposed slopes (1.5:1.0 in H:V) without a facing.

INSTRUMENTATION

Various measurement systems were installed in order to observe the detailed behaviours of the pier and the abutment, as shown in Figs. 5(a) and 5(b). The behaviours were continuously monitored by means of an automated measurement system throughout this project (for more than five years by the end of the full-scale loading tests).

Displacements

To measure the vertical compression of the backfill of the pier, four stainless steel reference plates were arranged at the bottom of the backfill. Brass rods covered with PVC pipes were connected to the reference plates and extended upwards to the level of the displacement transducers attached to the top reaction block. Two sets of the same type were installed to the abutment. The average compressions of the pier and the abutment reported in this paper were obtained from these measurements.

Tie Rod Tension, Earth Pressure and Inclination

The tie rod tension of the pier was measured by using tension meters consisting of electric resistance strain gages (ers-gages) that were attached to the surfaces of the tie rods. Two strain-gage-type earth pressure gages of

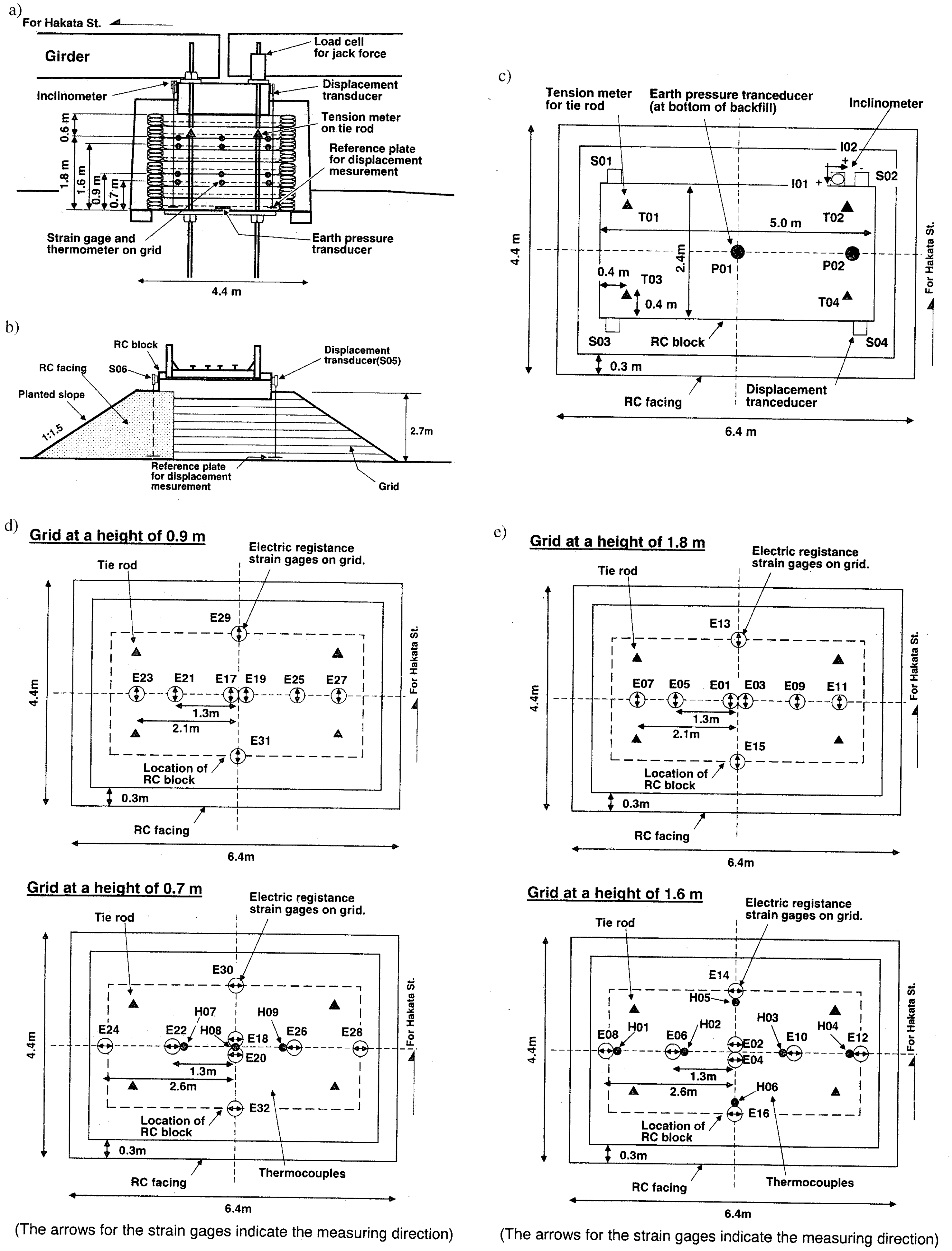

Fig. 5. Instrumentation for measurement: a) elevation view of the pier, b) elevation view of the abutment, c) plan view of the pier and d) and e) plan views at several levels of the pier (the arrows for the strain gages indicate the measuring direction)

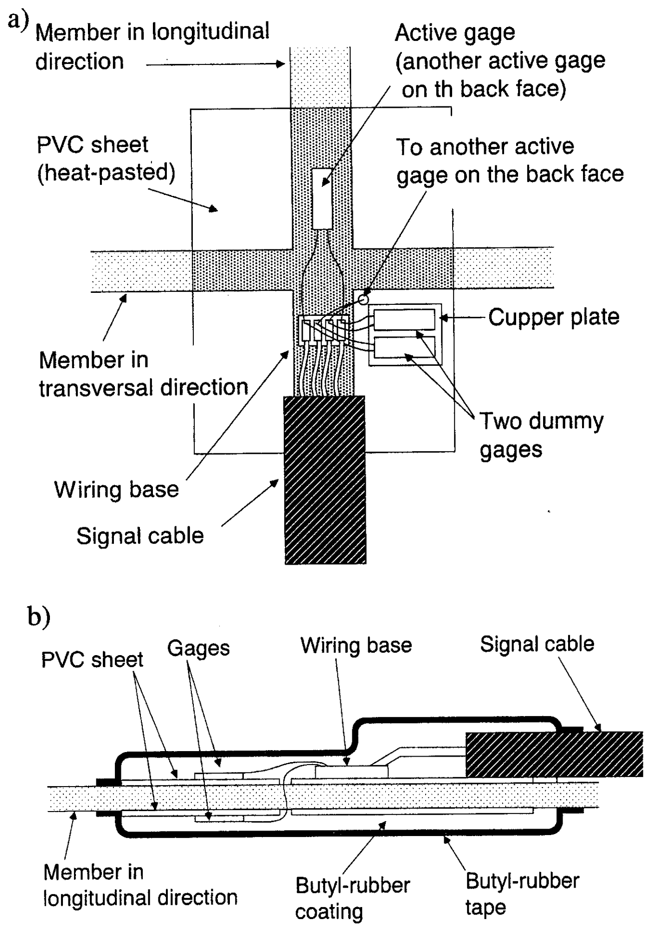

Fig. 6. Details of the measurement system for geogrid strain

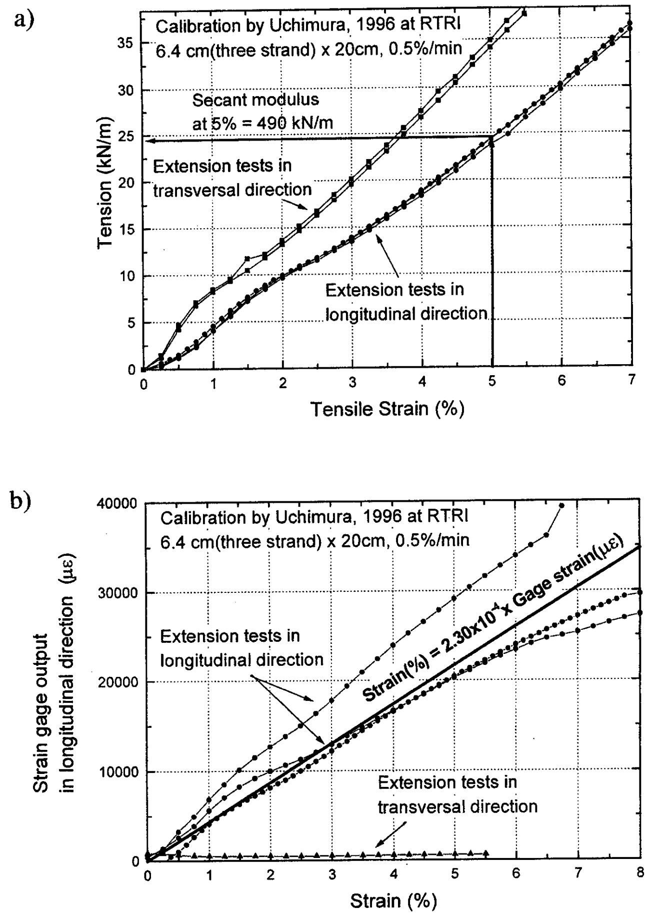

Fig. 7. Results from calibration tests on the geogrid: a) tensile load vs. tensile strain and b) gage output vs. tensile strain in two orthogonal directions

20 cm in diameter were placed at the center (P01) and near one set of tie rods (P02) in the bottom layer of the backfill of the pier. An inclinometer having two components in two orthogonal directions was set on one side of the top reaction block of the pier.

Geogrid Strains

Tensile strains activated in the reinforcement layers arranged in the backfill of the pier were measured at 32 points by using ers-gages attached to the geogrid. Full-bridges of ers-gages were formed as shown in Fig. 6(a) so that only normal strains developing in the longitudinal (or principal) direction of geogrid members, independently from those developing in the transversal direction, could be measured. To measure normal strains that were not affected by bending strains of reinforcement, a pair of active gages was attached respectively on the opposite faces of the longitudinal member of geogrid, while a pair of dummy gages were attached to a rigid cupper plate. The ers-gages and associated wire and other parts were embedded together in a water-proof coating covered with a special tape (*see* Fig. 6(b)).

The pier had a three-dimensional shape, unlike usual geogrid-reinforced soil retaining walls that essentially follow plane strain conditions. Thus, the grid reinforcement tended to deform in both the longitudinal and transversal directions. The longitudinal direction of the grid was either in parallel or orthogonal to the bridge axis alternatively layer by layer in the backfill. Therefore, normal strains in the direction parallel to the bridge axis were measured at half of the measurement points, while those in the direction orthogonal to the bridge axis were at the other half.

Figure 7 shows the results from calibration tests performed on the same type of geogrid as used for the pier. In these tests, geogrid specimens were extended in either the longitudinal or transversal direction while the normal strains activated in the longitudinal direction were measured. Figure 7(a) shows the relationships between; a) the tensile strain activated in the longitudinal direction and the tensile load applied in the longitudinal direction; and b) the tensile strain activated in the transversal direction and the tensile load applied in the transversal direction. It may be seen that the stiffness is larger in the transversal direction than in the longitudinal direction, while the relationships are nearly linear in both directions within the range of strain examined in these tests. The relationship in the longitudinal direction was used to deduce the tensile force acting in the geogrid from strains measured in the backfill of the pier.

Figure 7(b) shows the relationships between; a) the output of the ers-gages measuring tensile strains in the longitudinal direction and the tensile strain in the longitudinal direction; and b) the output of the ers-gages measuring tensile strains in the longitudinal direction and the tensile strain activated in the transversal direction. It may be seen that the ers-gages measuring tensile strains in the longitudinal direction did not respond to tensile strains activated in the transversal direction. This result validates the measuring method employed in the present study. The correction factor between the output of the ers-gages and the actual tensile strain in the longitudinal direction was obtained by linear-fitting of the average curve of the three relations indicated in the figure.

Temperature

Nine thermo-couples were installed inside the backfill of the pier, adjacent to the measurement points of geogrid strain (denoted as H in Figs. 5(d) and 5(e)). Another thermo-couple was set outside the backfill to measure the outside air temperature. The measurement of temperature was considered imperative, because the output of the ers-gages measuring geogrid strains is rather sensitive to changes in the temperature. The significance of this problem was confirmed by daily and seasonally systematic variations in the output of the ers-gages installed in the backfill of the pier. This problem could be attributed to the fact that each cupper plate, to which a pair of dummy gages was attached, has a thermal expansion coefficient that is noticeably higher than that of the geogrid material. A series of calibration tests were performed, in which a full bridge of ers-gages that were attached to the grid as in the actual case was cyclically heated and cooled. The following empirical relation was obtained from the test results:

$$\varepsilon_{\mathrm{corrected,20°C}}[\%] = \varepsilon_{\mathrm{measured}}[\%] - 0.003(T-20) \quad (1)$$

where $\varepsilon_{\mathrm{corrected,20°C}}$ is the tensile strain corrected to 20 degrees in Celsius; $\varepsilon_{\mathrm{measured}}$ is the normal strain measured with the ers-gage; and T is the temperature in Celsius measured with a thermo-couple set adjacent to the respective ers-gage. Geogrid strains corrected based on Eq. (1) are presented in this paper.

PRELOADING AND PRESTRESSING

Preloading Procedures

For a period of ten weekdays, preload was applied by using four hydraulic jacks (Fig. 3(a) and Photos. 4 and 7). As the preloading work was allowed only during daytime due to a restraint at the site, the total net preloading period was 72 hours. During nights and weekends, the tie rods were fixed to the top reaction block leaving the backfill under prestressed conditions.

Figure 8 shows the relationships between the total tie rod tension and the vertical compression of the backfill for the pier. Figures 9 and 12 show the detail and the whole of the time histories of the total tie rod tension and the compression of the backfill. In Fig. 12, the time history of the compression of the backfill of the abutment is

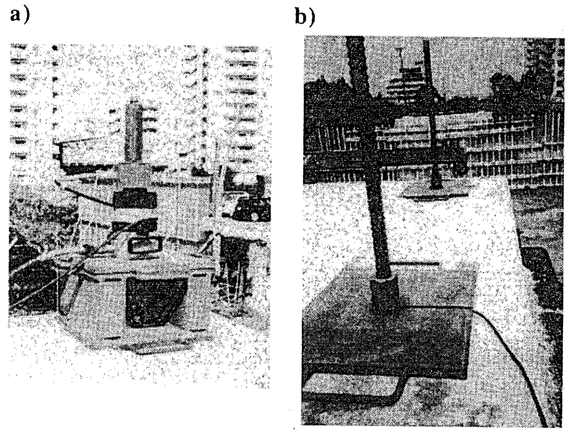

Photo. 7. a) Jack system for preloading and b) nut system for prestressing (pier)

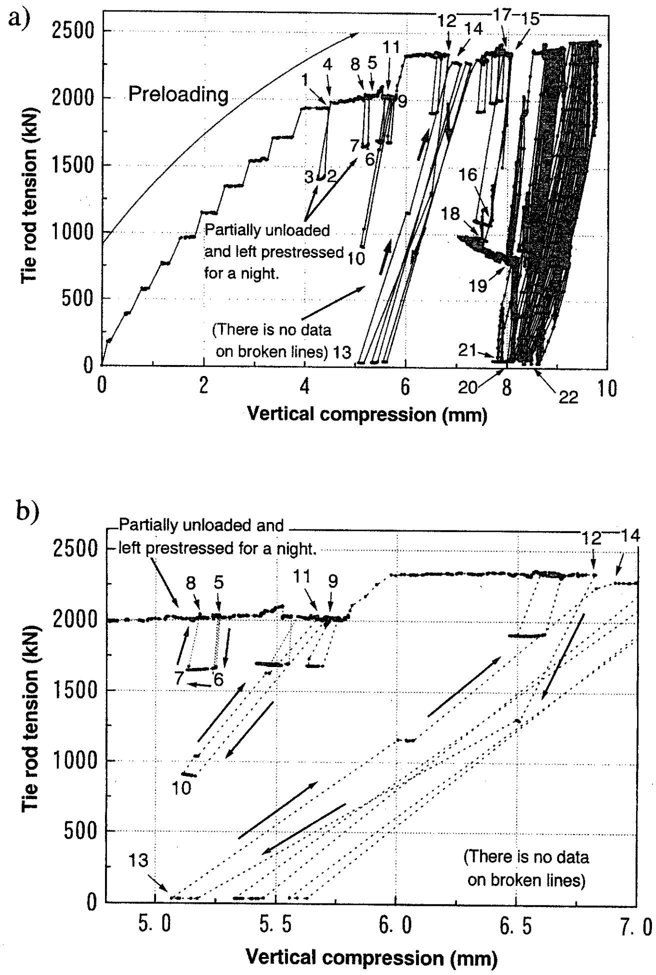

Fig. 8. a) Relationship between tie rod tension and settlement of the pier and b) its magnified figure (the numerals presented in this figure correspond to those presented in Figs. 9 through 12)

also presented. During the preloading work, the vertical load activated on the crest of the backfill is equal to the sum of the total tie rod tension and the weight of the top reaction block. During bridge in service, the total load acting on the crest of the backfill is equal to the sum of the total tie rod tension, the weight of the top reaction

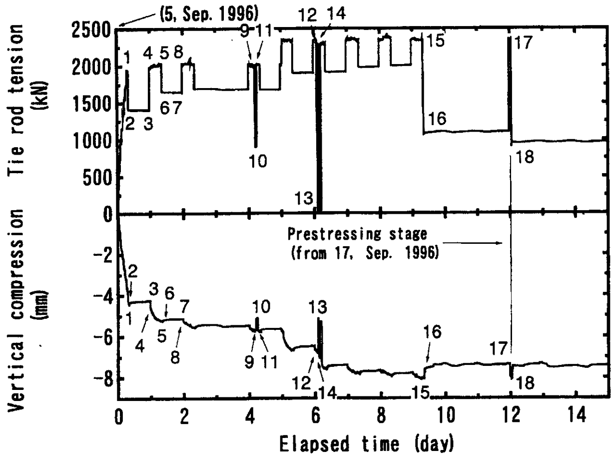

Fig. 9. Details of the time histories of total tie rod tension and vertical compression of the pier during preloading

block, a half of the total weight of the two girders, and the train load. The following is the details of the preloading process.

1) In the first day (5 September, 1996), the vertical load was increased step by step up to 1,960 kN (from the start of loading to stage 1 in Figs. 8, 9 and 12). In each loading step, a load increment of 196 kN was applied within two minutes or less and then the load was kept constant for 30 or 60 minutes.
2) In the fifth day, the load was decreased to 905 kN (stage 10), followed by reloading to the previous load level (stage 11).
3) During nights and weekends, the tie rods were fixed to the top reaction block leaving the backfill under prestressed conditions (e.g. between stages 2 and 3, between stages 6 and 7).
4) In the sixth day, the load was increased to 2,350 kN and kept constant (between stages 11 and 12).
5) In the seventh day, the load was decreased to zero (stage 13), followed by reloading to the previous load (stage 14).
6) In the eighth and ninth days, preloading at the maximum load level was continued until stage 15. A total compression of the backfill until stage 15 (the end of preloading stage in the ninth day) was about 8 mm.
7) In the tenth day, the load was decreased to about 1,100 kN (stage 16). Then the backfill was left under prestressed conditions for three days with the top of the tie rods being fixed to the top reaction block.
8) Finally, the load was increased to 2,350 kN (stage 17), which was maintained for three hours. Then, the load was decreased again to 950 kN (stage 18). Subsequently, the backfill was left under prestressed conditions until the end of the service of this bridge (until stage 19 for about four and a half years).

The rebound and re-compression of the backfill that took place during a cycle of unloading and reloading with a load amplitude of 1,400 kN applied in the last day (between stages 17 and 18) were nearly the same and equal to only 0.4 mm. This small value and high recoverability of deformation already indicated a very high stiffness and nearly elastic deformation characteristics of the backfill. The data after stage 19 in Figs. 8 and 12 were obtained from full-scale loading tests that were performed after the end of service of the bridge.

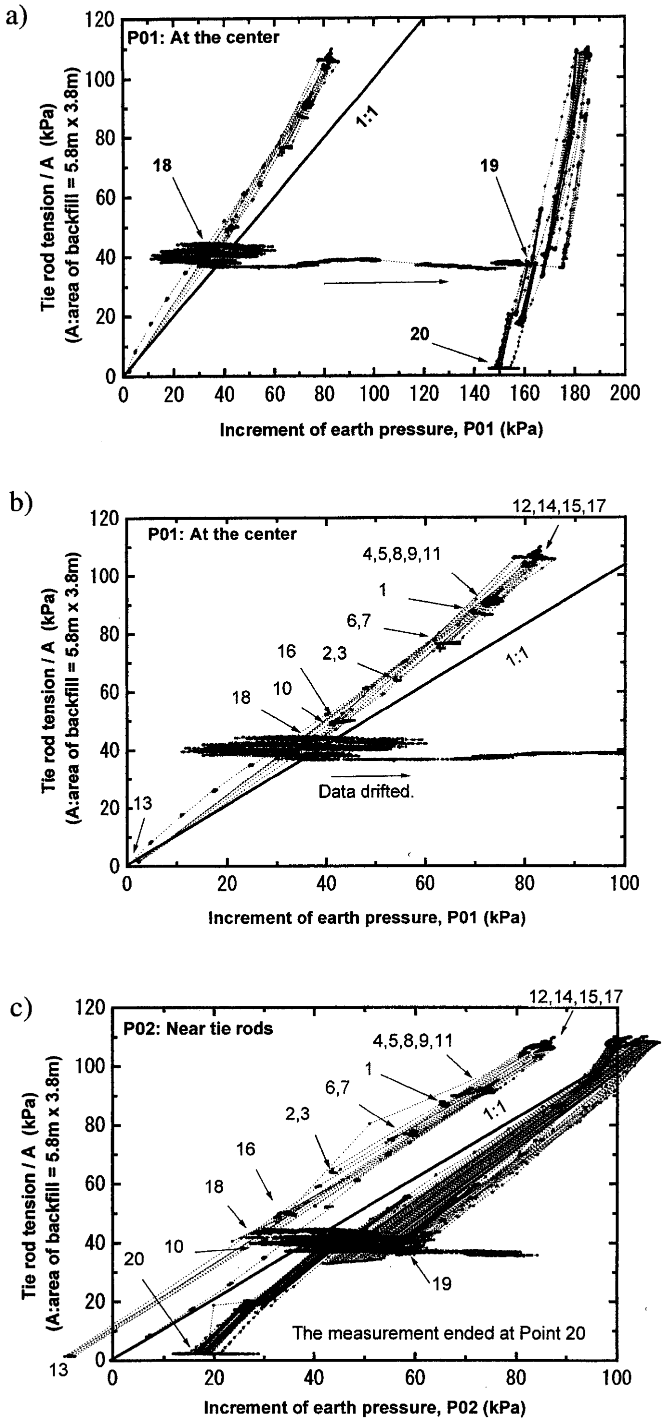

Fig. 10. a) and c) relationships between earth pressure P01 and P02 and tie rod and b) zoom-up during stages 1–18 of a)

Stiffness of the Backfill

It may be seen from Fig. 8(b) that the average stiffness of the backfill when reloaded from stage 10 to state 11 is noticeably smaller than the value when reloaded in a

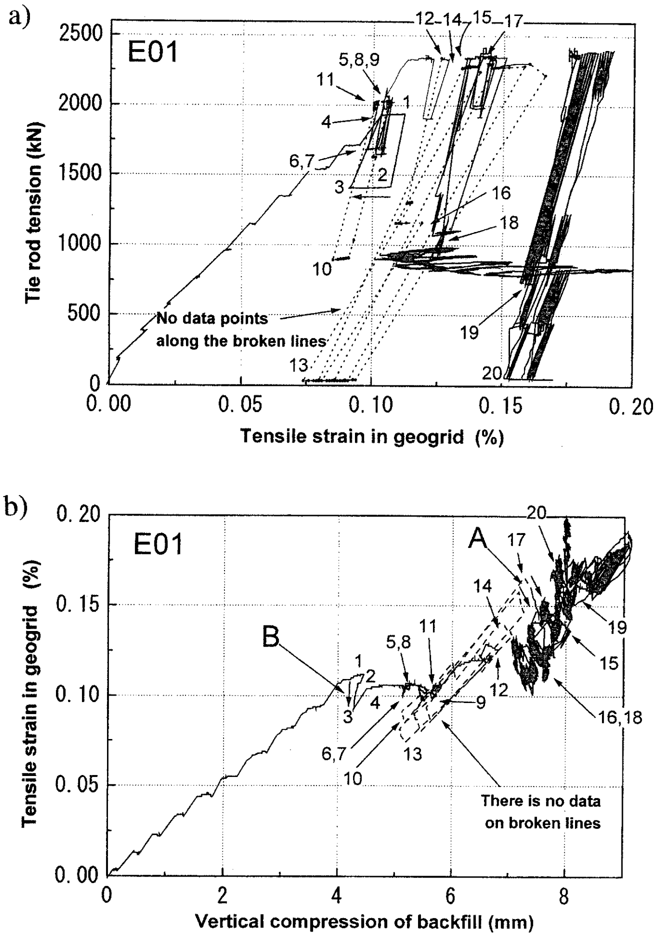

Fig. 11. Relationships between: a) grid strain (E01) and tie rod tension and b) vertical compression of the backfill and grid strain (E01), pier P1

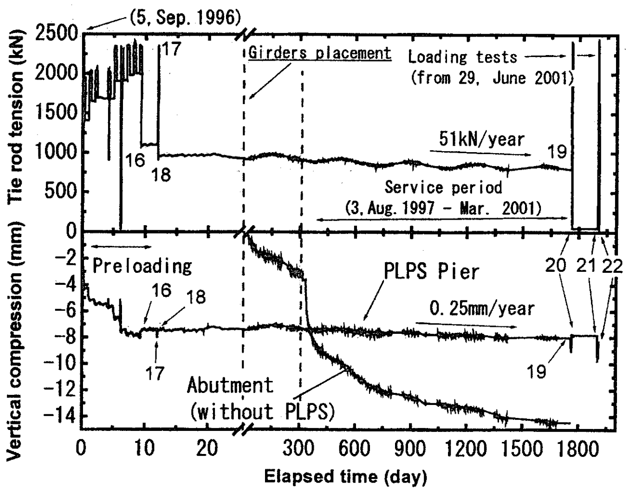

Fig. 12. Long-term time histories of total tie rod tension in the pier and vertical compression of the pier and abutment

higher load level (e.g., reloading from stage 7 to stage 8). Moreover, the average stiffness observed when the backfill was reloaded from the nearly zero load (from stage 13) is much smaller than the above. This trend is due likely to the effects of swelling and associated softening of the backfill caused by a reduction in the vertical stress in the backfill, which became particularly large at low stress levels. This fact indicates the paramount importance of maintaining a sufficiently high prestress in the backfill (Tatsuoka, 1997). This point was confirmed by behaviour during the full-scale unloading and reloading tests with a large load amplitude (stages 19 to 22) performed after the end of service of the bridge.

Earth Pressure in the Backfill

Figures 10(a) and (c) show the relationships between the average vertical pressure in the backfill by the tie rod tension and the earth pressure increment measured at the center (P01 in Fig. 5) and near the tie rods (P02), both located in the bottom layer of the backfill, for the entire period of the present study. Figure 10(b) is the zoom-up of Fig. 10(a), presenting the relation during the preloading stage. The average pressure shown in the vertical axis of these figures is equal to the total tie rod tension divided by the total cross-sectional area (5.8 m × 3.8 m) of the pier backfill. It does not include the vertical pressure in the backfill due to the weight of the top reaction block and the girders, which is equal to 20 kPa. The earth pressure increment in the horizontal axis of these figures is defined to be zero at the start of the preloading stage. The following trends until the end of the preloading stage (until stage 18) may be seen from these figures:

1) The relationship is rather linear, while the earth pressure increment is similar to the average applied pressure.
2) The readings of the earth pressure gages P1 and P2 are nearly the same. These results show that the stress distribution in the backfill was reasonably uniform and the bottom reaction block made by cement-mixing-in-place functioned as a nearly rigid body.

The readings of the earth pressure gages tended to increase after stage 18 (the end of the preloading stage). A sudden and large drift in the earth pressure of gage P01 is probably due to damage to the gage by wetting during a flood that took place on the 1,027th day from the beginning of preloading (29 June, 1999). One third of the height of the pier was submerged during this flood. On the other hand, the total drift in the reading of the pressure gage P02 is much smaller. This drift may be inevitable during a relatively long period of observation; however the authors could not find a specific cause for this drift.

Strains in the Reinforcement

Figure 11(a) shows the relationships between the strains (positive in extension) in the geogrid at E01 (*see* Fig. 5(e)) and the total tie rod tension (i.e., the major part of the vertical load applied to the backfill) for the pier. Figure 11(b) shows the relationships between the compression of the backfill and the geogrid strain (E01). The geogrid strains presented in these figures have been corrected for temperature changes based on Eq. (1). The following trends of behaviour may be seen from these figures:

1) The tensile strain in the geogrid generally increased

with an increase in the vertical compression of the backfill (the primary loading stage before stage 1, for example). This is the basic mechanism of tensile-reinforcement that the lateral expansion of the backfill is restrained by tensile reinforcement.

2) When the vertical compression of the backfill was increasing under constant load just after the load had been increased to the ever-largest level, the tensile strain in the geogrid did not tend to increase, or even decreased, with time. The behaviour denoted by the letter A in Fig. 11(b) is typical of the above. Although it was subtle, the same trend was observed during each step of the primary loading stage before stage 1. It seems that the backfill exhibited creep compressive deformations in the lateral direction due to the tensile force activated in the reinforcement. It is important to note that this trend of behaviour is opposite to the usual design concept for GRS retaining walls where reinforcement layers arranged in the backfill exhibit creep tensile strains under static load conditions. It seems that this design concept is too conservative with respect to the creep deformation of geogrid.
3) The creep deformation in the geogrid was compressive at the unloaded stages from stage 2 to stage 3, denoted by the letter B, where the vertical load applied to the backfill was kept nearly constant. This trend of behaviour would also be due to the viscous properties of the backfill, as discussed above. This fact indicates that noticeable creep tensile strains may not develop in the geogrid under prestressed conditions as long as relatively large residual compression of the backfill does not take place, for example, by long-term cyclic loading.

The trends described above confirmed observations of similar behaviours of full-scale test walls of PLPS geosynthetic-reinforced backfill constructed at Chiba Experiment Station, Institute of Industrial Science, University of Tokyo (Uchimura et al., 1996).

LONG-TERM BEHAVIOUR

Following the preloading stage, full-height rigid lightly steel-reinforced concrete facings (30 cm-thick) were cast-in-place directly on the wrapped-around wall faces of the pier. Subsequently, a pair of metal bridge girders, each weighing 211 kN, were placed on the pier and abutment on the 25th day from the beginning of preloading (30 September, 1996). Then, the behaviours of the pier and the abutment were observed continuously for about five years. The bridge was left without being open to service for about ten months after completion. On the 317th day from the beginning of preloading (19 July, 1997), a diesel locomotive of 637 kN weight passed 6 times over the bridge for inspection. The residual compression of the backfill of the pier by this event was 0.02 mm, while that of the backfill of the abutment was much larger, equal to 0.52 mm. The bridge was opened to service on the 332nd day from the beginning of preloading (3 August, 1997). On average 124 trains, each consisting of two to four coaches, passed over the bridge every day. Each coach weighed 30 to 40 tons excluding the weight of passengers.

Deformations of the Pier and Abutment and Tie Rod Tension

Figure 12 shows the full time histories of the vertical compression of the pier and the abutment and the tie rod tension in the pier for the five years. The vertical compression of the backfill of the pier is defined to be zero at the start of preloading work. On the other hand, the vertical compression of the abutment backfill was measured from immediately after the girder was placed. So, the instantaneous compression upon the placement of the girder is excluded in the vertical compression of the abutment backfill presented in Fig. 12. The following trends of behaviour may be seen from Fig. 12:

1) The total compression of the pier backfill that took place under the prestressed conditions for the first ten months, between the completion of the bridge and the opening of service, was essentially zero. Corresponding to the above, the total reduction in the tie rod tension was also essentially zero. This extremely high performance could be attributed to a relatively large compression, equal to about 8 mm that took place during the preloading stage.
2) The rate of the compression of the pier backfill after opening to service was also very small, equal to only 0.25 mm/year on the average. Corresponding to the above, the decreasing rate of the tie rod tension was also very small, equal to 51 kN/year. These rates of change were small enough for the temporary use of the pier for the planned term of about four years. Based on the results from small-scale model tests performed in the laboratory, Shinoda et al. (1999, 2003) suggested using "a ratchet connection system" to fix the top ends of the tie rods to the top reaction block. By using this system, the prestress could be maintained to a sufficiently high level even when relatively large compression takes place in the backfill during a long life time of the structure.
3) The compression of the abutment backfill for the first ten months after completion was much larger, equal to about 3 mm, despite the fact that no live load had been applied. This relatively large creep compression was caused by the self-weight of the abutment and half of the weight of a single girder. The compression rate became much larger after opening to service, and the compression continued for more than three years until the end of service. Interestingly, the long-term settlement at the crest of the embankment immediately in the back of the abutment was similar to that at the crest of the reinforced backfill of the abutment. Therefore, there was not a serious problem with this long-term compression of the backfill of the abutment.

A sharp contrast between the behaviours of the pier and the abutment indicates that the preloading and

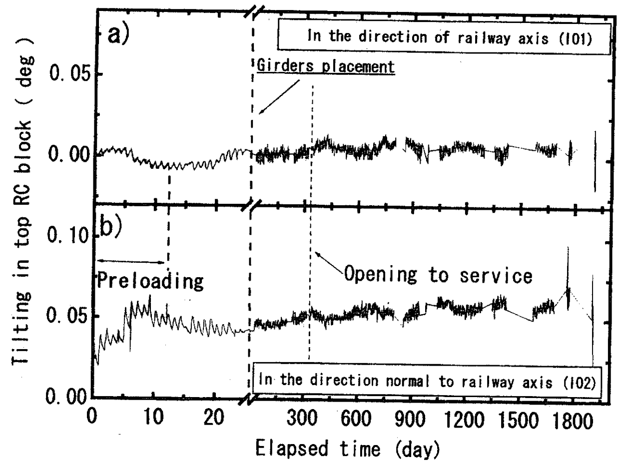

Fig. 13. **Long-term time histories of the inclination of the top reaction block of the pier**

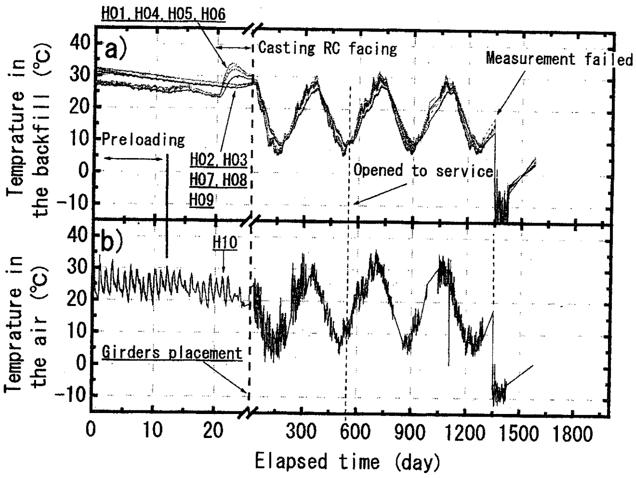

Fig. 14. **Long-term time histories of the temperature inside and outside the backfill of the pier**

prestressing procedure was very effective to decrease long-term vertical compression of the reinforced backfill caused by static load as well as live load by traffic. Note that the average dry density of the backfill of the abutment was slightly higher than that of the pier. The advantages of the PLPS reinforced soil method described above were re-confirmed by results from small-scale model tests in the laboratory simulating this full-scale behaviour (Shinoda et al., 1999, 2003).

The tie rod tension and the compression of the pier noticeably fluctuated over the long period of observation. As the fluctuation was rather systematic and cyclical in a period of one year, it is likely due to annual changes of temperature, as described later in this paper.

Tilting of the Top Reaction Block

Figure 13 shows the time histories of the tilting of the top reaction block in the bridge axis direction and its orthogonal direction. The behaviour after the completion of the bridge until the end of service was consistently very stable. Some tilting was observed in the direction normal to the bridge axis (I02 in Fig. 5(c)) during the preloading stage (Fig. 13(b)). It is likely due to some bending deformation exhibited by the RC top reaction block upon the application of the preload by means of the tie rods placed near the both ends of the block, because the tilting transducer was set near the edge of the RC block.

Temperature

Figure 14(a) shows the time histories of the temperature measured at nine points inside the backfill of the pier (*see* Fig. 5), while Fig. 14(b) shows that of the air temperature. The following trends of behaviour could be seen:

1) During the preloading stage, the temperature inside the backfill gradually dropped at a rather constant rate. This was due to an initial difference between the initial temperature of the backfill soil at compaction under heavy sunshine and the average air temperature.
2) Upon the casting-in-place of the RC facings, the soil temperatures adjacent to the facings (H01, H04, H05 and H06) showed a temporary rise due to the hydration heat of the concrete. Some time later, these temperatures inside the backfill became nearly the same with each other and the outside air temperature (H10).
3) Although the daily change of the temperature inside the backfill was much smaller than that in the outside air temperature, the average temperatures for every period of several days inside and outside the backfill were nearly the same, showing a systematic annual variation over this long period of observation. This variation is reflected in several physical quantities measured at the pier. For example, when the average temperature became lower in the winter, the measured height of the backfill of the pier increased and the measured tie rod tension became higher (Fig. 12). This could be the true behaviour; but, at least partly, was also due to an electrical drift in the strain amplifiers caused by these temperature changes.

Grid Strains

Figure 15(a) shows the time histories of the average of the extensional strains in the geogrid in the bridge direction measured at 16 points (Fig. 5). Figure 15(b) shows the average of the normal strains in the direction normal to the bridge axis measured at other 16 points. These strains have been corrected for temperature changes based on Eq. (1). The following trends of behaviour may be seen:

1) The strains in the geogrid noticeably changed during the preloading stage and the full-scale loading tests after the end of service. Compared to the above, the extensional strains in the geogrid increased only at a very low rate under prestressed conditions during the subsequent four years until the end of service. It is likely that this increase was not due to the creep deformation of the geogrid, but associated with the gradual compression of the backfill. This is because the overall relationship between the geogrid strain

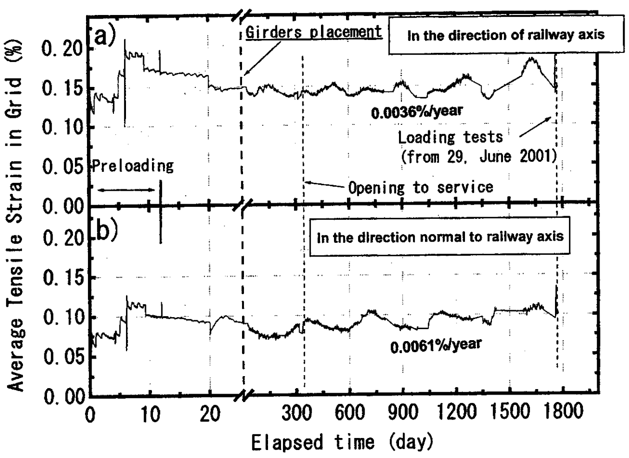

Fig. 15. Long-term time histories of grid extension (averaged in each direction and corrected for temperature changes) of the pier

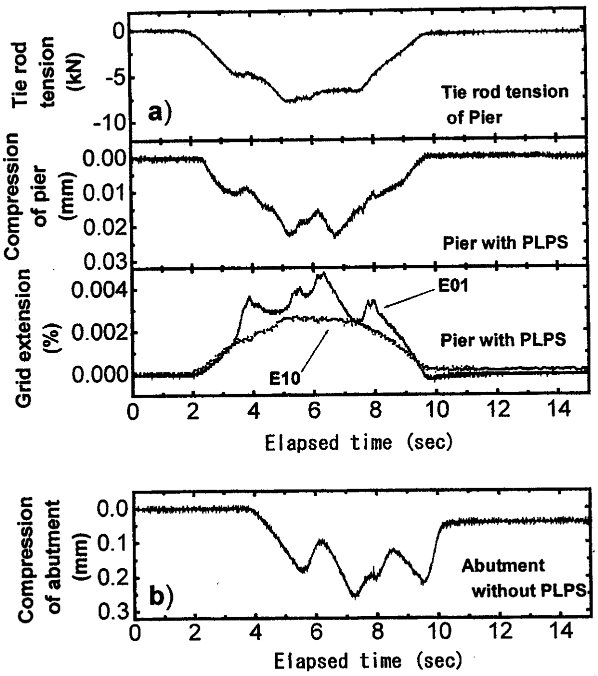

Fig. 16. Behaviours of a) the pier and b) the abutment during the first train passing in service (6:15 AM, 3 August, 1997)

and the compression of the backfill during this long-period (i.e., the behaviour between stages 18 and 19 in Fig. 11) is consistent with the overall trend. The creep deformation was not an actual serious problem in this case.

2) It is difficult to determine whether some small annual variations in the geogrid strains seen in these figures were either actual behaviours or just behaviour recorded due to imperfect corrections for temperature changes, or both.
3) The largest strain in the geogrid that developed during this project was on the order of only 0.2%. Based on the results of the calibration tests presented in Fig. 7(a), this strain value corresponds to a tensile force of 980 N/m in the geogrid per width, which is equivalent to a confining pressure of 6.5 kPa per unit wall area. In this analysis, the material viscous properties of the geogrid are not taken into account. These relatively small strain and stress values indicate that the design of the bridge pier was extremely conservative with respect to the rupture failure of reinforcement. It is true however that it would not be possible to make the backfill very stiff by applying a very high preload and keep a relatively high prestress if the backfill were not reinforced. Further study will be necessary on the optimum economical yet sufficient arrangement of reinforcement.

TRANSIENT BEHAVIOUR DURING TRAIN PASSINGS

Figure 16 shows the time histories of several physical quantities measured at the pier and the abutment during the first train passing in service (the 332nd day; 3 August, 1997). The train consisted of two coaches, each weighing 353 kN (not including the weight of passengers). The following trends of behaviour may be seen:

1) The maximum compression of the backfill of the pier was extremely small, equal to about 0.02 mm, which is equivalent to a compressive strain of about 0.001% of the backfill (Fig. 16(a)). This extremely small strain corresponded to the nearly elastic behaviour of the backfill without essentially zero residual compression. The limit strain for the elastic behaviour of a gravel is estimated to be in the order of 0.001% in general (Tatsuoka et al., 1999a and b). The strain of the backfill above is similar to the elastic limit strain, which is consistent with the highly elastic behaviours of the pier backfill.
2) The tie rod tension in the reinforcement temporarily decreased responding to the compression of the backfill of the pier. The load activated on the backfill became smaller by this amount when compared to the value in the case without the prestressing system.
3) The tensile strains activated in the grid were very small and recoverable, corresponding to the behaviour of the backfill of the pier. Apparently, the major cause for this very high performance of the backfill is not an increase in the reinforcement force during train passing, but the highly stiff and highly elastic deformation characteristics of the backfill. Note, however, that these significant properties of backfill could be realised by applying sufficiently large preload and prestress, which became feasible with a help of reinforcement.
4) In comparison, the compression of the backfill of the abutment was equal to about 0.2 mm (Fig. 16(b)). Although this deformation did not endanger the train passing at all, it was about 10 times larger than that of the pier backfill. The equivalent vertical strain was of the order of 0.01%, which was much larger than the strain limit for elastic behaviour. It is known that noticeable residual strain can develop when strain cycles of this order repeat in a mass of granular

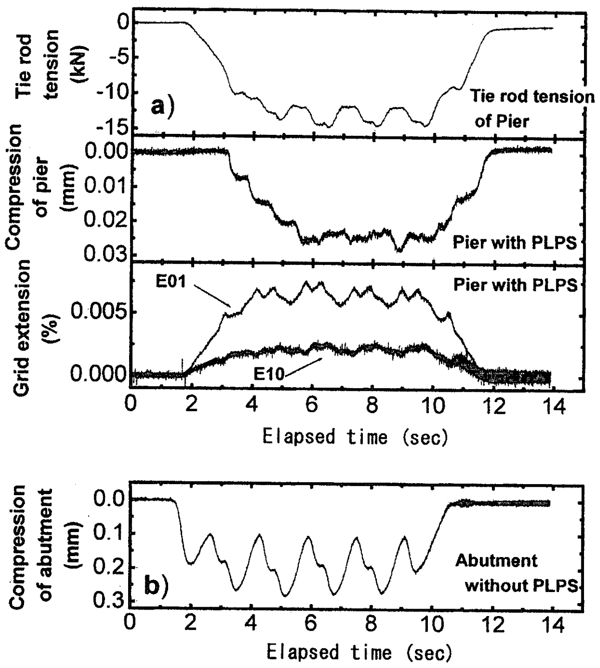

Fig. 17. Behaviours of a) the pier and b) the abutment during a train passing after 2 years of service (8:10 AM, 16 July, 1999)

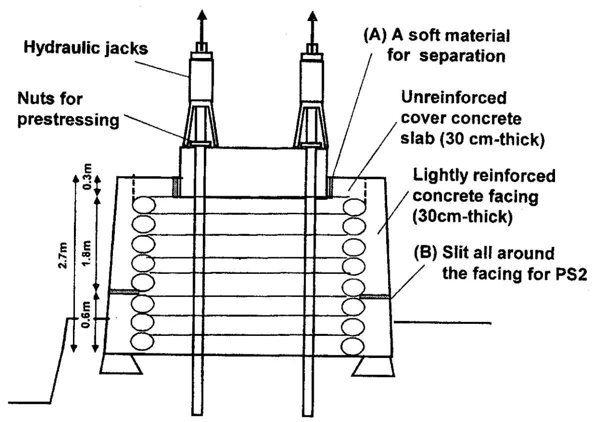

Fig. 18. Set up of full-scale loading tests PS1 and PS2, performed in 2001

material (e.g., Tatsuoka et al., 1999a). In fact, a noticeable residual deformation remained after the passing of the first train (Fig. 16(b)).

Figure 17 shows the time histories, similar to Fig. 16, which were obtained two years after opening to service. Nearly the same trends of behaviour Fig. 16 may be seen, showing that the pier and the abutment had been stable and their properties essentially did not change after service for two years.

The different behaviours of the pier and abutment clearly show significant advantages of the preloading and prestressing procedures in making the deformation of reinforced backfill subjected to long-term cyclic loading very small. More specifically, it is necessary to keep the transient compressive strain in the backfill as small as 0.001%, or less, so that the backfill could deform essentially elastically without showing intolerable residual compression by long-term traffic load. The relevant preloading and prestressing procedures can realize such conditions.

Elastic Stiffness of the Backfill under Prestressed Conditions

Based on the readings of the earth pressure, the vertical stress induced by train passing described above was around 7 kPa. As the amplitude of the lateral extension of the geogrid due to the traffic load was of the same order as that of the vertical compression of the pier, the ratio between the amplitude of the vertical compression and the vertical stress, $E_v = 7\ \text{kPa}/0.001\% = 700\ \text{MPa}$, can be considered to be nearly the same as the Young's modulus of the pier.

On the other hand, based on the triaxial compression test results on specimens of the backfill material, whose void ratio was 0.38 and the dry density was 1.95 g/cm^3, E_v can be expressed as:

$$E_v = c \cdot (\sigma_v / p_a)^m \tag{2}$$

where σ_v is the instantaneous vertical stress (kPa); $p_a = 98$ kPa is a constant; $c = 610$ MPa is a coefficient that is a function of the specimen density; and $m = 0.63$ is the exponent. The average void ratio, e, of the backfill of the pier obtained by the sand replacement method at demolishing the pier was equal to about 0.32. So, the value of c obtained for $e = 0.32$ when based on a void ratio function $f(e) = (2.17 - e)^2/(1 + e)$ is equal to be 680 MPa. The value of the vertical stress σ_v at the bottom of the backfill under the prestressed conditions was of the order of 115 kPa. By substituting these values of c and σ_v into Eq. (2), we obtain a value of E_v equal to 740 MPa. This value is very similar to the back-calculated value from the field full-scale behaviours presented above.

The analysis above indicates that a very high performance of the pier during train passing, which was realised by the preloading and prestressing process, should not be surprising, and could be predicted based on the stress-strain properties of the backfill material and given stress conditions of the backfill.

FULL-SCALE LOADING TESTS

In July and November 2001, after the end of service, full-scale vertical loading tests of the pier were performed twice as follows:

1) The metal girders were removed in June of 2001.
2) *Test PS1*: In July 2001 (from the 1,758th day after the beginning of the preloading), the first series of post-service vertical loading tests were performed. Figure 18 shows the set up of testing. The same set of hydraulic jacks as used for preloading and prestressing the backfill of the pier at the construction stage was used. Figure 19 shows the time histories of the applied vertical load (i.e., the total tie rod tension) and the compression of the backfill. The relationship between the total load applied to the top reaction block and the vertical compression of the backfill

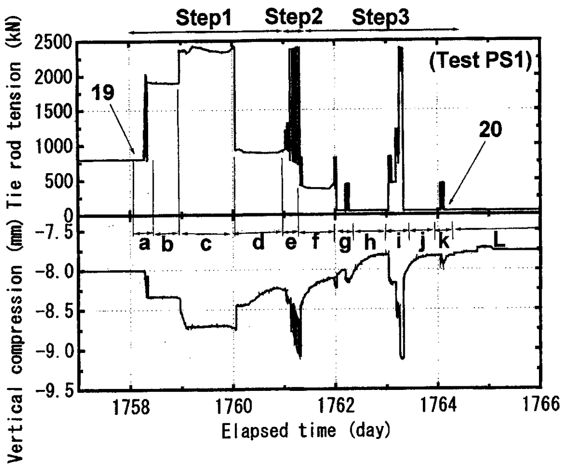

Fig. 19. Time histories of applied vertical load (i.e., total tie rod tension) and compression of the backfill from test PS1

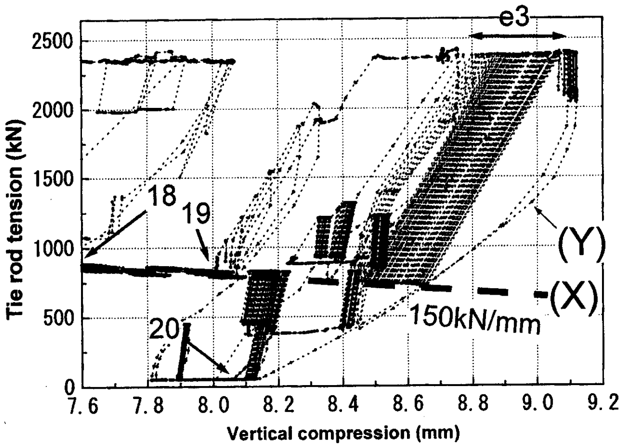

Fig. 20. Zoom-up of Fig. 8 showing the results from test PS1

from test PS1 are presented in Fig. 8. Figure 20 shows a zoom-up of the part of Fig. 8 showing the results from test PS1.

a) *Step 1*: The vertical load was increased while applying cyclic load toward about 2,000 kN, and was maintained for 0.6 days. Subsequently, the vertical load was increased to about 2,350 kN, and maintained for 1 day. Then, the vertical load was decreased to about 900 kN, and maintained for 1 day.
b) *Step 2*: 120 cycles of vertical load with amplitude of 400 kN was applied. Subsequently, the jack force was repeatedly changed from 0 to 2,350 kN for 120 cycles (e3 in Fig. 20). Note that the load activated on the top reaction block did not become zero even when the applied cyclic load became zero due to the presence of the nuts (*see* Photo. 7).
c) *Step 3*: The nuts were not removed just before this step so that the load applied to the backfill could be nearly zero. A cycle of global unloading to zero and reloading was applied with small reload/unload cycles at intermediate stages during unloading (f, g, i and k in Fig. 19).

3) *Test PS2*: On the 1,896th day after the beginning of the preloading (14 November, 2001), lateral slits were made in the facings along the periphery, as denoted by B in Fig. 18, to make the vertical load supported by the facings as small as possible. Then, cyclic loading tests as step 1 in test PS1 were performed (stages 21 to 22 in Figs. 8 and 12).

In this paper, only parts of the results from these tests are reported. The details of the results from these loading tests, including the time history of the applied load and the vertical compression of the test PS2, will be reported in the companion paper (Uchimura et al., 2004).

The following trends of behaviour may be seen from Figs. 8 and 20:

1) The stiffness of the backfill had not changed noticeably after about five years.
2) The curve denoted by the letter X in Fig. 20 represents the relationship between the load applied to the top reaction block and the compression of the backfill at the moment when the jack force was zero, obtained from step 2 loading. The slope of the relation X is the same as that of the relation between steps 18 and 19 for a period of about five years from August 1996 to July 2001. The results from test PS1 confirmed the trend of behaviour from the long-term observations of the full-scale behaviour. The slope of these relations is equal to 150 kN/mm, which represents the total stiffness of the four tie rods. This stiffness is smaller than the stiffness of the backfill under prestressed conditions by a factor of around one fiftieth. Because of this relatively low stiffness of the tie rod system, the rate of the reduction in the tie rod tension due to residual compression of the backfill was relatively low. The tie rods that were used in this project are one of the stiffest PC (prestressed concrete) tie rods that are available in the market. So, it can be concluded that it is quite feasible to prepare tie rods that are soft enough to keep the reduction rate of tie rod tension small enough for a long term as far as the residual compression of the PLPS reinforced backfill is not very large.
3) The total residual compression of the backfill by step 2 cyclic loading was only 0.2 mm (e3 in Fig. 20), despite a relatively large number of cycles with a relatively large load amplitude. This high performance would be a result of the preloading history and some residual compression developed for a long-term period of five years and during step 1 loading.
4) The load-compression relations denoted as e3 in Fig. 20 were obtained by connecting the maximum and minimum load states with linear segments recorded in step 2 loading. On the other hand, the unloading curve denoted by the letter Y, which was obtained from step 3, consists of continuous data points. The two relations are therefore consistent to each other. It may be seen from the unload curve Y that the swelling rate of the backfill increased significantly with decrease in the vertical load.
5) During otherwise global loading/unloading during

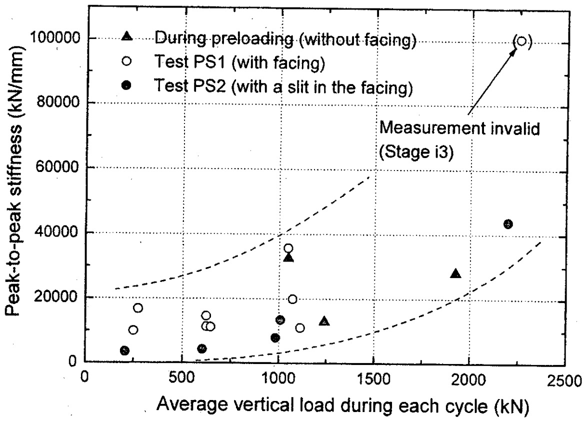

Fig. 21. Stiffness of the pier backfill from small amplitude (400 kN) cyclic loading tests during preloading and loading tests SP1 and SP2

the preloading stage and in tests PS1 and PS2, 120 small reload/unload cycles with an amplitude of 400 kN were applied several times. Figure 21 shows the relationships between the average stiffness and the load level obtained from those cyclic loading tests. Despite a large scatter in the data, a trend of decrease in the stiffness of the backfill with a decrease in the vertical stress is clear. This result confirms the importance of prestressing to maintain a high stiffness of the backfill under working load conditions.

CONCLUSIONS

The first prototype preloaded and prestressed geogrid-reinforced soil pier was constructed for a railway bridge, and its behaviour was carefully observed during and after construction. The following conclusions can be derived from the observations reported above:

1) The transient and long-term residual deformation of the geogrid-reinforced backfill of the pier was very small. The deformation was substantially smaller than that of another geogrid-reinforced backfill of a bridge abutment that was constructed without preloading and prestressing at an adjacent place at the same time. This fact shows that the preloading and prestressing procedure is very efficient to restrain transient and residual deformation of the backfill subjected to cyclic load by traffic load for a long duration.
2) The high performance of the pier was due to the highly stiff and elastic deformation characteristics of the backfill that was achieved by the preloading and prestressing procedures. The stiffness of the backfill back-calculated from the full-scale behaviour was consistent with the value estimated based on results from laboratory stress-strain tests on the backfill material.
3) The reduction rate of prestress against residual compression of backfill becomes smaller with a decrease in the stiffness of the tie rod system under otherwise the same conditions. This requirement could be satisfied by using an ordinary type of PC steel tie rod available in the market.

ACKNOWLEDGEMENTS

The authors deeply appreciate help in this long-term research project given by Messrs. Koga, T., Tsuru, H., Maeda, T., Hyodo, K. and Kaise, H. of Kyushu Railway Company, Messrs. Kojima, K. and Watanabe, K. of Railway Technical Research Institute, Dr. Koseki, J. and Mr. Sato, T. of Institute of Industrial Science, University of Tokyo, Mr. Maruyama, D. of University of Tokyo, Mr. Nakamura, H. of Nihon University, and Messrs. Sugimura, Y., Ishihara, M., Shinoda, M., Hirakawa, D., and Nakarai, K. of the graduate school of the University of Tokyo.

REFERENCES

1) Adams, M. (1997): Performance of a prestrained geosynthetic reinforced soil bridge pier, *Mechanically Stabilized Backfill*, (ed. by Wu, J. T. H.), Balkema, 35–53.
2) Ketchart, K. and Wu, J. T. H. (1997): Performance of geosynthetic-reinforced soil bridge pier and abutment, Denver, Colorado, USA, *Mechanically Stabilized Backfill* (ed. by Wu, J. T. H.), Balkema, 101–116.
3) Shinoda, M., Uchimura, T., Maruyama, N. and Tatsuoka, F. (1999): Effects of preloading and prestressing on the vertical stiffness of GRS structure, *Proc. of 11th Asian Regional Conf. on SMGE*, Seoul, **1**, 419–422.
4) Shinoda, M., Uchimura, T. and Tatsuoka, F. (2003): Increasing the stiffness of mechanically reinforced backfill by preloading and prestressing, *Soils and Foundations*, **43**(1), 75–92.
5) Tatsuoka, F. (1997): Why is PS required in addition to PL, *Mechanical Stabilized Backfill* (ed. by Wu, J. T. H.), Balkema, 445–448.
6) Tatsuoka, F., Jardine, R. J., Lo Presti, D., Di Benedetto, H. and Kodaka, T. (1999): Characterizing the pre-failure deformation properties of geomaterials, Theme Lecture for the Plenary Session No. 1, *Proc. of XIV IC on SMFE*, Hamburg, September 1997, **4**, 2129–2164.
7) Tatsuoka, F., Lo Presti, D. C. F. and Kohata, Y. (1995): Deformation characteristics of soils and soft rocks under monotonic and cyclic loads and their relationships, SOA Report, *Proc. of the 3rd Int. Conf. on Recent Advances in Geotec. Earthquake Engrg. and Soil Dynamics*, (ed. by Prakash), St. Luois, **2**, 851–879.
8) Tatsuoka, F., Modoni, G., Jiang, G. L., Anh Dan, L. Q., Flora, A., Matsushita, M. and Koseki, J. (1999): Stress-strain behaviour at small strains of unbound granular materials and its laboratory tests, Keynote Lecture, *Proc. of Workshop on Modelling and Advanced Testing for Unbound Granular Materials*, January 21 and 22, 1999, Lisboa (ed. by Correia), Balkema, 17–61.
9) Tatsuoka, F., Tateyama, M., Uchimura, T. and Koseki, J. (1997): Geosynthetic-reinforced soil retaining walls as important permanent structures, 1996–1997 Mercer Lecture, *Geosynthetic Int.*, **4**(2), 81–136.
10) Tatsuoka, F., Uchimura, T. and Tateyama, M. (1997): Preloaded and prestressed reinforced soil, *Soils and Foundations*, **37**(3), 79–94.
11) Uchimura, T., Shinoda, M., Tatsuoka, F. and Tateyama, M. (2001): Performance of PLPS geosynthetic-reinforced soil structure against working and seismic loads, *Proc. 15th ICSMFE*, Istanbul, **2**, 1633–1636.
12) Uchimura, T., Tamura, Y., Tateyama, M., Tanaka, I. and Tatsuoka, F. (2004): Vertical and horizontal loading tests on full-scale preloaded and prestressed geogrid-reinforced soil structures,

Soils and Foundations, (under preparation).
13) Uchimura, T., Tatsuoka, F., Sato, T., Tateyama, M. and Tamura, Y. (1996): Performance of preloaded and prestressed geosynthetic-reinforced soil, *Int. Symp. on Earth Reinforcement*, (eds. by Ochiai et al.), Balkema, Fukuoka, Japan, **1**, 537–542.
14) Uchimura, T., Tatsuoka, F., Tateyama, M. and Koga, T. (1998): Preloaded-prestressed geogrid-reinforced soil bridge pier, *Proc. the 6th Int. Conf. on Geosynthetics*, Atlanta, **2**, 565–572.