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VERTICAL AND HORIZONTAL LOADING TESTS ON FULL-SCALE PRELOADED AND PRESTRESSED GEOGRID-REINFORCED SOIL STRUCTURES

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ABSTRACT

The performance of a preloaded and prestressed (PLPS) geogrid-reinforced soil pier constructed for a railway was evaluated by vertical loading tests performed after a service period of about 3.5 years. The performance was compared with those at the preloading stage during construction as well as during service. The test results confirmed that the high performance of the pier (i.e., very small transient deformation during train passing and substantially small residual deformation by long-term traffic load during service) can be attributed to the preloading and prestressing procedure. Horizontal loading tests were performed simultaneously on the top RC blocks on the preloaded-prestressed pier and the geogrid-reinforced soil abutment, which was constructed without the PLPS procedure for the same bridge. The results showed that the pier was substantially more stable against over-turning moment and horizontal shear load than the abutment, indicating that the PLPS procedure is also very effective to achieve a high seismic stability.

Key words: cyclic loading, deformation, earthquake resistant, preloading, (prestressing), reinforced soil (**IGC:** E12/K14)

INTRODUCTION

The applications of the reinforced soil structure technology have been extended to construct permanent critical civil engineering structures, such as bridge abutments supporting not only the self-weight of backfill but also the dead load of superstructure and the live load from traffic. For example, Abu-Hejuleh et al. (2002) reported high performance of a pair of geogrid-reinforced soil bridge abutments directly supporting a pair of highway bridge girders, 34.5 m-long and 34.5 m-wide. Adams (1997) and Ketchart and Wu (1997) constructed full-scale models of geosynthetic-reinforced soil bridge pier and performed loading tests on them.

Tatsuoka et al. (1997) 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. Tatsuoka et al. (1997) and Uchimura et al. (1996) performed full scale model tests on PLPS reinforced soil embankments to confirm and discuss its feasibility and mechanisms. Shinoda et al. (2003a, b) performed a series of model tests in the laboratory to demonstrate the advantages of the preloading and prestressing procedure in achieving a very high performance of geosynthetic-reinforced soil structures against not only long-term traffic load but also severe seismic load.

Uchimura et al. (2003) reported case history of construction and long-term performance of a preloaded and prestressed (PLPS) geogrid-reinforced soil pier for a railway bridge, Maidashi Bridge (Fig. 1). The bridge was constructed in July, 1996 in Fukuoka City, Kyushu, Japan. The geogrid-reinforced backfill of the pier was preloaded in the vertical direction by using four tie rods installed inside the backfill and then continuously prestressed during a period of about five years. A geogrid-reinforced soil bridge abutment was also constructed in the same way as the pier for the same bridge, which was neither preloaded nor prestressed. A pair of simple beam girders was placed on the completed pier and the abutment in October, 1996, and the bridge was opened to service from the beginning of August, 1997 until the end of March, 2001. The residual compression of the PLPS geogrid-reinforced soil bridge pier observed during service was essentially negligible when compared to noticeable residual compression of the geogrid-reinforced

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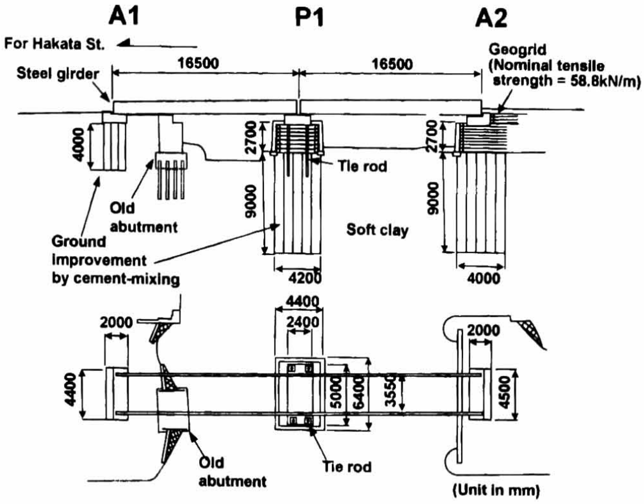


Fig. 1. General view of Maidashi bridge (Uchimura et al., 2003)

soil bridge abutment. This fact indicated that the preloading and prestressing procedure could keep the stiffness of the backfill very high and the residual deformation very small.

To confirm the findings from the full-sale behaviour and the laboratory model tests, the authors performed full-scale vertical cyclic loading tests on the PLPS geogrid-reinforced soil pier in July and November, 2001 after a service of about 3.5 years. Horizontal loading tests on the pier and the abutment of geosynthetic-reinforced soil were also performed in July, 2001 by applying the same horizontal load to the top reinforced concrete (RC) blocks, which supported the girders, of the two structures. In this paper, the performance of the PLPS geogrid-reinforced soil pier and abutment observed during these full-scale loading tests are reported and the effects of the preloading and prestressing procedure are evaluated based on the test results.

OUTLINE OF PLPS GEOGRID-REINFORCED SOIL PIER

The PLPS geogrid-reinforced soil pier, denoted P1 in Fig. 1, supported 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. 2(a)). The design dead load by the girder weight was 196 kN while the design live load by the weight of locomotives, including impact load, was 1,280 kN. The four steel tie rods, vertically installed in the backfill, had a nominal yield tensile force of 1,034 kN per rod. Their lower ends were vertically inserted and anchored into the cement-mixed sub-ground.

The backfill soil was a well-graded gravel of crushed sandstone ($D_{max} = 30$ mm, $D_{50} = 8$–11 mm and $U_c =$ 4.0–4.3), which was compacted to a dry density of 1.91–2.17 g/cm^3. These density values were measured when the backfill was demolished, which correspond to 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 pier was constructed with a help of gravel-filled bags that were stacked at the shoulder of each gravel layer along the periphery of the backfill. The bags were wrapped around with the respective geogrid reinforcement sheet.

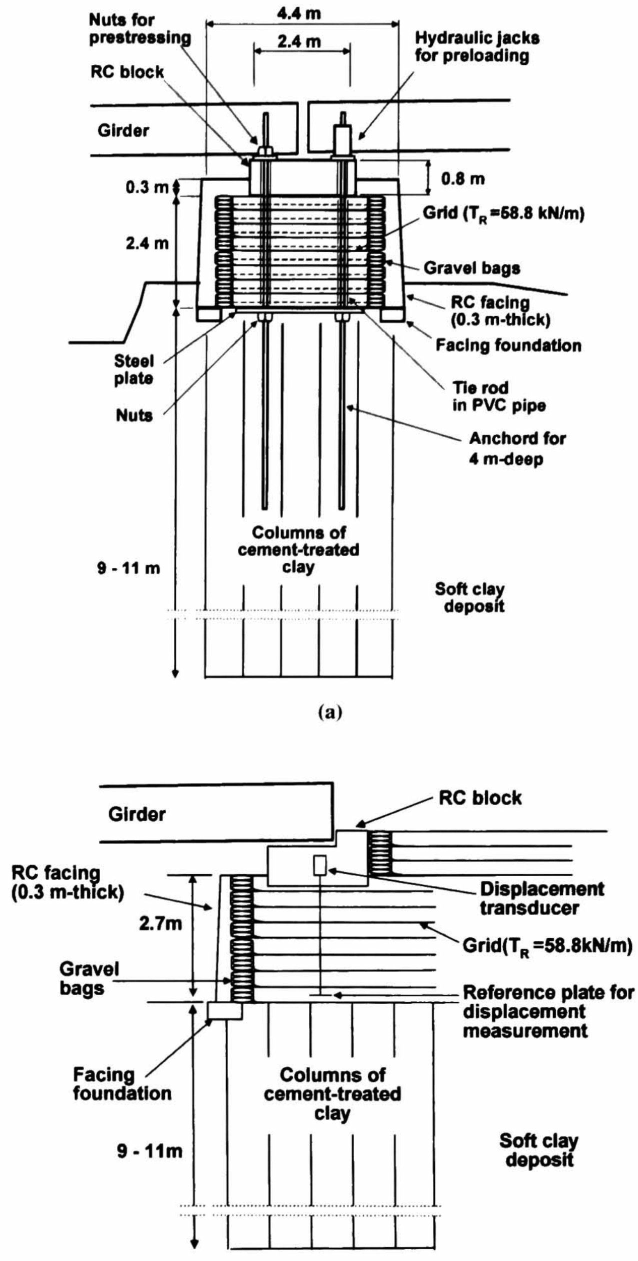


Fig. 2. (a) PLPS reinforced soil pier and (b) reinforced soil retaining wall abutment

The reinforcement was a geogrid made of polyvinyl alcohol, coated with polyvinyl chloride (PVC), having a nominal rupture strength of 73.5 kN/m and a nominal stiffness value of 1,050 kN/m at strains less than 1.0%. The average vertical spacing between the reinforcement layers was 15 cm.

After the backfill was completed, an RC block to support the girders was constructed, which also worked as the top reaction block to apply preload and prestress to

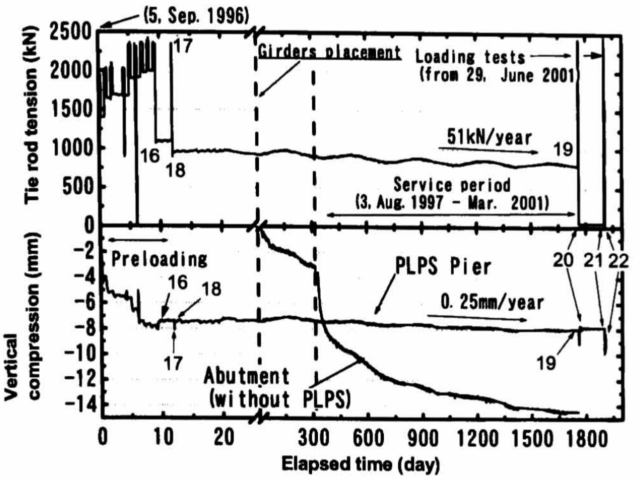


Fig. 3. Long-term time histories of total tie rod tension in the pier and vertical compressions of the pier and the abutment (the numerals presented in this figure correspond those presented in Figs. 4, 5 and 7) (Uchimura et al., 2003)

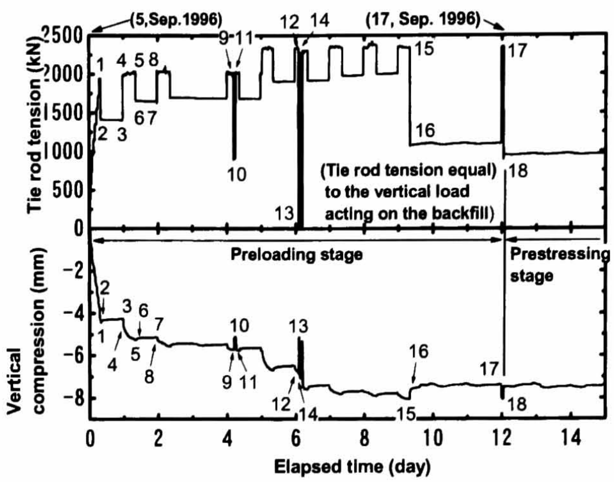


Fig. 4. Detailed time histories of total tie rod tension and vertical compression of the pier during preloading (Uchimura et al., 2003)

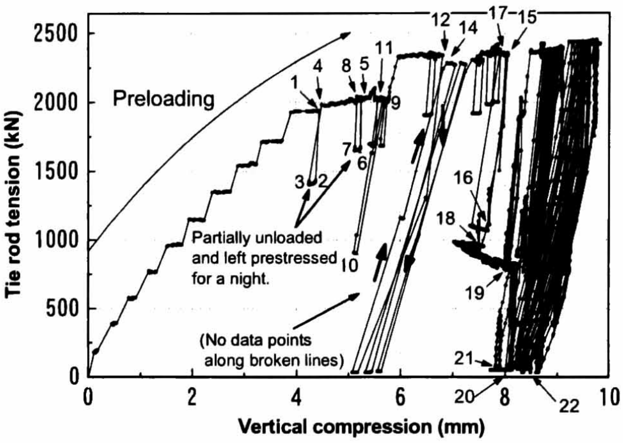


Fig. 5. Relationship between applied vertical load (i.e., total tie rod tension) and compression of the pier (Uchimura et al., 2003)

the backfill. After the PLPS procedure, 30 cm-thick full-height rigid facings of lightly steel-reinforced concrete were cast-in-place on the four wall faces, as illustrated in Fig. 2(a).

The abutment, denoted as A2 in Fig. 1 and described in Fig. 2(b), was a conventional type geogrid-reinforced soil retaining wall, which was constructed as one of the two abutments of the bridge. The abutment was constructed in the same way as the pier, except that it had only one wall face and was neither preloaded nor prestressed. The dry density of the gravel backfill measured when it was demolished ranged 2.08–2.19 g/cm^3, which is even slightly higher than that of the backfill of the pier. The backfill of the abutment was reinforced with the same type of geogrid as used for the pier at a vertical spacing of 30 cm. The sides of the backfill consisted of exposed slopes (1.5:1.0 in H:V) without facing. It should be noted that the vertical spacing of the geogrid in the pier A1 was a half of that of the abutment A2. Uchimura and Mizuhashi (2005) showed that the difference in the vertical spacing of the reinforcement layers does not affect the quasi-elastic Young's modulus of the reinforced backfill under cyclic loading with small amplitude, e.g. traffic load applied to the pier and the abutment. However, quantitative evaluation of its effects on the residual and/or creep deformations during a long-term are issues of the future. As in the case of Maidashi Bridge, the authors consider that the difference between the long-term behaviours of the pier and the abutment cannot be due to the reinforcement spacing only.

More details of the pier and the abutment and their instrumentations are reported in Uchimura et al. (2003).

Figure 3 shows the full time histories of the tie rod tension and the vertical compression of the backfill of the pier P1 as well as the vertical compression of the backfill of the abutment A2. The total vertical load acting on the top RC block of the pier is the sum of the total tie rod tension, the weight of the girder and the traffic load. The vertical compressions of the backfills of the pier and abutment were obtained from the settlements of the top RC blocks relative to the bottom of the backfill. The origin of the vertical compression of the pier was at the starting of the preloading stage, while that of the abutment is defined immediately after the girder was placed. So the vertical compression of the abutment shown in Fig. 3 does not include the creep compression before putting the girder and the instantaneous compression by the girder weight.

PRELOADING AND PRESTRESSING

Preloading and Prestressing

The backfill of the pier was preloaded by using four hydraulic jacks. Figure 4 shows the detailed time histories of the tie rod tension and vertical compression of the backfill during the preloading stage. Figure 5 shows the full relationships between the tie rod tension and the vertical compression of the backfill during the preloading stage as well as the service period and subsequent vertical loading tests. The preloading stage ended at stage 18 as indicated in these figures. The total net preloading period

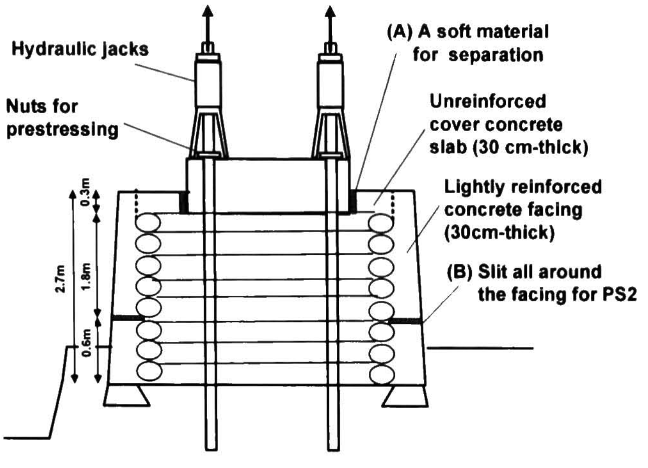


Fig. 6. Set up of full-scale vertical loading tests PS1 and PS2 of the pier: The concrete facing did not exist at the preloading stage of construction

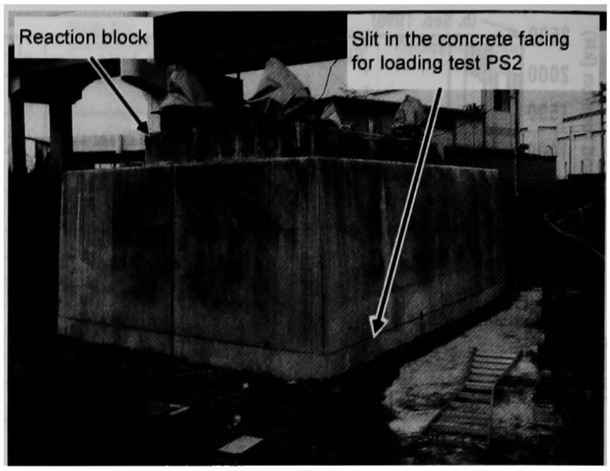


Photo 1. Full-scale vertical loading test PS1 and PS2 of the pier

was 72 hours compared to 12 days in total that were spent at the preloading stage, because the active preloading procedure was allowed to perform only during daytime due to a restraint at the site. During nights and weekends, the top ends of the tie rods were fixed to the top reaction block leaving the backfill under prestressed conditions. After the vertical load was reduced to about 950 kN, the top ends of the tie rods were fixed to the top RC block before removing the hydraulic jacks. The maximum compression during the preloading period was 8 mm.

After casting the facing, the space between the top RC block and the top of the RC facing on the crest of the backfill was covered with a 30 cm-thick unreinforced concrete slab. A soft joint material was placed around the side faces of the top RC block, as denoted by (A) in Fig. 6, so that essentially all the vertical load acting on the RC block is transmitted to the backfill. Then, a pair of simple beam steel girders, each weighing 211 kN, was placed on the pier and abutment in the 26th day. Ten months later, the bridge was opened to service, which continued for 3.5 years (stages 18 through 19 in Figs. 3 and 5). On average 124 trains consisting of two to six coaches passed over the bridge every day. Every coach weighed 300 to 400 kN without including the weight of passengers. The average rate of the compression of the pier backfill was only 0.25 mm/year and, correspondingly, the decreasing rate of the total tie rod tension was as small as 51 kN/year (Fig. 4). These low rates were more than sufficient for the temporary use of the pier for about four years.

VERTICAL LOADING TESTS

Vertical Loading Test (PS1)

After the end of service of the bridge, the girders were removed in June, 2001. Subsequently, full-scale horizontal loading tests and then vertical loading tests were performed in July, 2001 to:

1) evaluate possible changes in the deformation characteristics of the backfill of the pier during the period of service for about 3.5 years; and
2) confirm the effects of preloading and prestressing on the deformation characteristics of the backfill when subjected to vertical and horizontal load.

The loading procedures consist of several components. Cyclic loads with amplitude of 400 kN, which is similar to the traffic load applied to the pier in service, were applied at various load level to observe the effects of the prestressing level. Sustained loads were also applied at various stages for a several hours to observe the effects of the preloading procedures and the prestressing level. Cyclic load with large amplitude were also applied to observe the dynamic deformation characteristics of the reinforced soil backfill. Every night, the pier were left fully unloaded, or in a prestressed condition with tension in the tie rods with nuts, because the loading system was not available.

Unlike the preloading stage, there existed lightly reinforced concrete facing cast-in-place around the backfill and an unreinforced concrete slab around the top reaction block as shown in Fig. 6 and Photo 1. As the concrete slab was mechanically separated by a soft joint material from the reaction block, the load applied on the reaction block was directly supported by the top surface of the backfill. The influence of lateral confinement due to the stiffness of the facing can be considered to be small enough, because each of the four facings has a joint line with a soft material in the vertical direction at the center to allow their small displacement. In addition, the gravel bags stacked around the backfill may be softer than the well-compacted backfill, reducing the effect of the facing stiffness.

The details of the vertical loading test PS1 (i.e., from stage 19 to stage 20 in Fig. 3), are as follows. Figures 7 and 8 show the details.

1) *Step 1:* The initial prestress at the start of test PS1 was 800 kN. The nuts were in use to fix the tie rods to the top RC block at steps 1 and 2. First, the backfill was vertically loaded and unloaded between 850 kN to 2,040 kN, while increasing the amplitudes from 120 kN to 1,240 kN (stage **a** in Figs. 7 and 8). A verti-

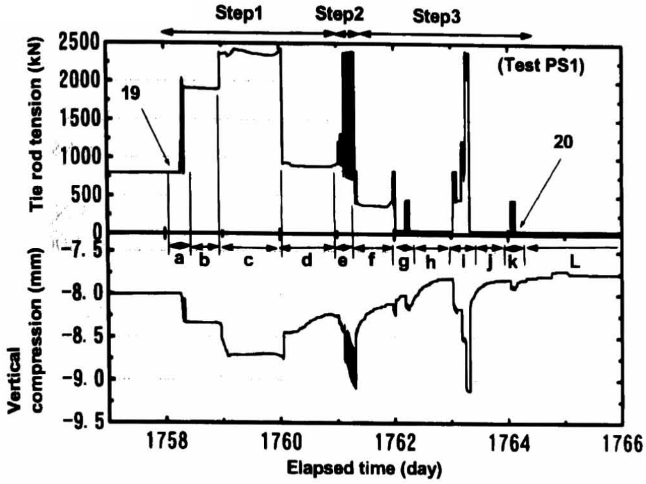


Fig. 7. Time histories of applied vertical load and compression of the pier, test PS1 (the letters presented in this figure correspond to those in Fig. 8)

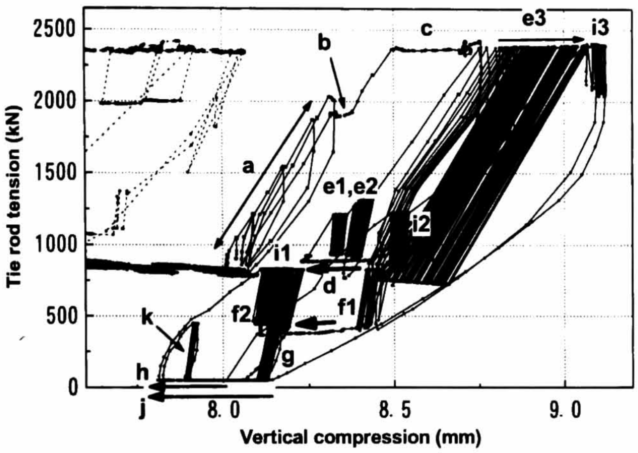


Fig. 8. Relationship between applied vertical load and settlement of the pier, test PS1

cal load of about 2,000 kN was then maintained for 14 hours (stage **b**). Subsequently, the vertical load was increased to about 2,400 kN, which was then maintained for one day by locking the hydraulic jacks (stage **c**). Then, the vertical load was decreased to about 900 kN, and the jacks were locked again for another day (stage **d**).

2) *Step 2:* A series of cyclic vertical loads consisting of; a) 50 cycles with an amplitude of 300 kN; b) 70 cycles with an amplitude of 400 kN; and c) subsequently 120 cycles with an amplitude of 2,350 kN were applied (stages **e1**, **e2** and **e3**).
3) *Step 3:* The nuts, which had been used to fix the tie rods to the top RC block, were removed. Then, the vertical load was decreased to 400 kN, and 50 cycles of cyclic vertical load with an amplitude of 400 kN were applied (stage **f1**). Then the jack loads were kept constant for 15 hours allowing the backfill to swell before 70 cycles of vertical load with amplitude of 400 kN (stage **f2**) were applied. Then, the load was

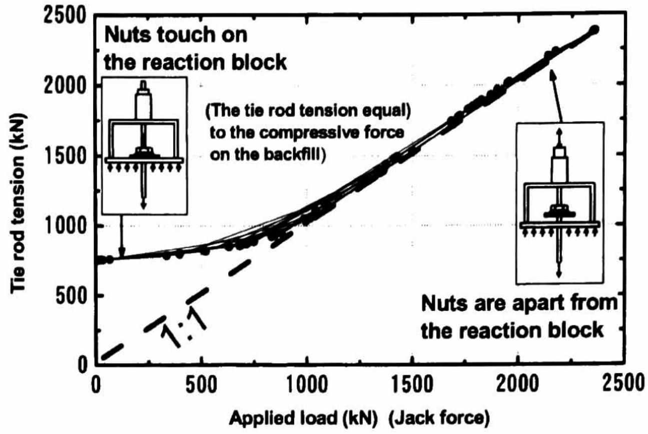


Fig. 9. Relationships between hydraulic jack force and tie rod tension of the PLPS GRS pier during loading stage e3 in test PS1

fully released to zero, and 120 cycles of vertical load with amplitude of 400 kN was applied (stage **g**). After leaving the backfill under fully unloaded conditions for 18 hours (stage **h**), one cycle of global loading to 2,350 kN and subsequent full unloading was applied with small reload/unload cycles applied at intermediate stages (stages **i1**, **i2** and **i3**). After leaving the backfill under fully unloaded conditions for 18 hours (stage **j**), 120 cycles of cyclic vertical load with an amplitude of 400 kN were applied again (stage **k**). Finally, the pier was left under fully unloaded conditions for 4.5 months (stage **L**).

The load-compression relations during the cyclic loading test (stage **e** in step **2**) in test PS1 are represented in Fig. 8 by linear segments connecting the maximum and minimum load states, except for the first and last cycles of each cyclic loading stage.

Figure 9 shows the relationships between the measured total tie rod tension, which is equal to the load applied to the crest of the backfill of the pier, and the total jack force measured by using load cells for the loading stage **e3**. This result is typical of those measured at steps 1 and 2, where the nuts were in use to fix the tie rods to the top RC block. When the jack load became less than about 750 kN, the contact of the nuts with the RC block started developing and the total tie rod tension became in equilibrium with the sum of the force acting at the nuts and the jack force. In this way, the tie rod tension was kept nearly constant despite the jack force becoming lower than 750 kN. That is, the vertical load acting at the crest of the backfill did not become lower than about 750 kN. On the other hand, when the jack force became higher than 750 kN, the nuts were pull up losing the contact with the top RC block, and the total tie rod tension became equal to the total jack force.

Vertical Loading Test (PS2)

The vertical loading test PS2 was performed in November, 2001.

Before starting the test, a lateral slit with a width of around 5 mm, as denoted by (B) in Fig. 6 and shown in

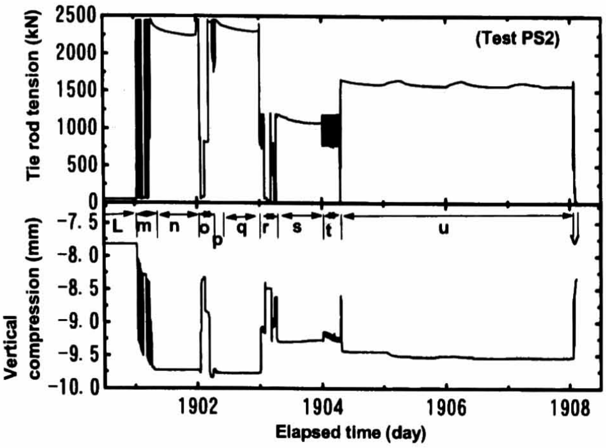


Fig. 10. Time histories of applied vertical load and compression of the pier, test PS2 (the letters presented in this figure correspond to those in Figs. 11)

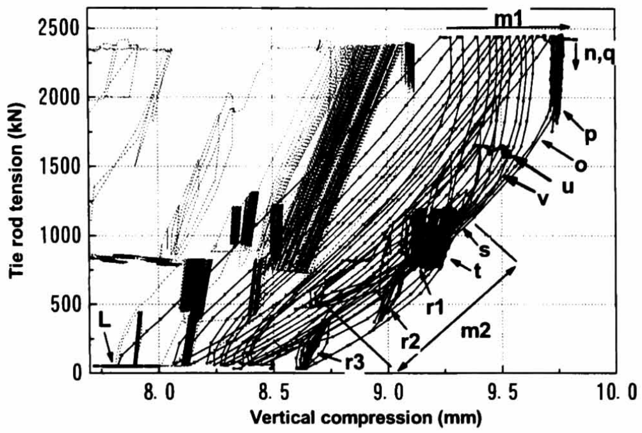


Fig. 11. Relationships between applied vertical load and compression of the pier, test PS2

Photo 1, was made along the periphery of the facing. This is in order to minimise the effects of stiffness of the facing in the vertical direction, while the effects of stiffness in the horizontal direction were already minimized by the vertical joint mentioned before. The nuts were not used in Test PS2. The details of the loading procedures (from stage 21 to stage 22 in Fig. 3) are as follows. They consist of the components almost similar to those of SP1, in order to observe the effects of slit in the facing, as well as fully unloading period for 5 months between SP1 and SP2. Figures 10 and 11 show the test results:

1) Vertical cyclic loads with amplitude of 2,350 kN were applied for 10 times (stage **m1**). Then, the load was decreased to 850 kN, and subsequently, load/unload cycles between 850 kN to 2,030 kN, with various load amplitudes ranging from 120 kN to 1,180 kN were applied (stage **m2**).
2) The vertical load was increased to 2,350 kN, and the jacks were locked for 17 hours (stage **n**). Then, one cycle of full unloading and reloading with amplitude of 2,350 kN was applied while performing creep load tests at intermediate stages where the vertical load was 0 kN, 880 kN and 2,350 kN for 40–100 minutes each (stage **o**).
3) Cyclic vertical loads with amplitude of 400 kN were applied for 120 times (stage **p**). Then, the vertical load was increased to 2,350 kN before the jacks were locked for 17 hours (stage **q**).
4) Cyclic vertical loads with amplitude of 400 kN were applied for 120 times at each load levels of 800 kN, 400 kN and 0 kN (stages **r1**, **r2** and **r3**). Then, the vertical load was increased to 1,200 kN, and the jacks were locked for 18 hours (stage **s**).
5) Cyclic vertical loads with amplitude of 400 kN for a range of 800–1,200 kN were applied for 100 times (stage **t**). In this stage, an interval for four minutes was given between consecutive cycles to simulate the actual traffic load which has long intervals between train passings while no interval was given for other cyclic loading states. However, any clear effects of the intervals were not observed.
6) The vertical load was increased to 1,600 kN, and the jacks were locked for 4 days (stage **u**). Finally, the vertical load was fully released to zero (stage **v**).

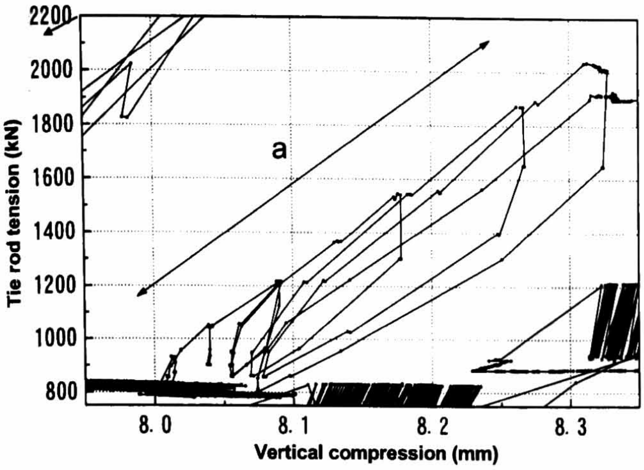
DISCUSSIONS ON VERTICAL LOADING TESTS

Stiffness of the Backfill

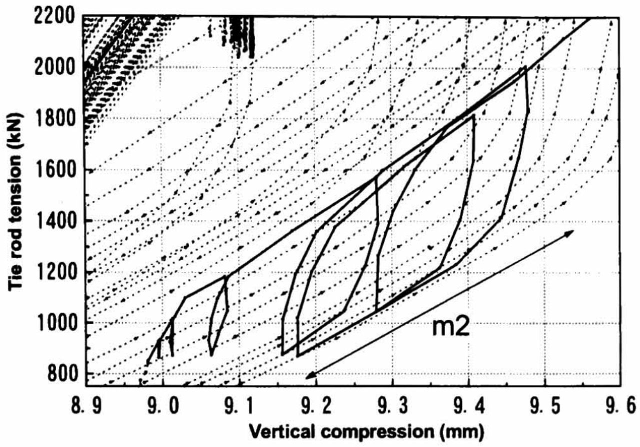
The backfill of the pier exhibited very high stiffness against cyclic vertical loads having relatively large amplitudes when compared to a low stiffness exhibited during the primary loading in the preloading procedure (up to stage **1**; Figs. 4 and 5). For example, the rebound and recompression of the backfill during a full unload/reload cycle with a load amplitude of 1,400 kN at the final stage of the preloading procedure (from stage 17 to stage 18 in Figs. 4 and 5) was only 0.4 mm.

Figure 12(a) shows the zoomed-up load-compression relationships at stage **a** in test PS1 (Fig. 8), where cyclic vertical loads with various amplitudes ranging from 120 kN to 1,180 kN were applied. Figure 12(b) shows the relationships between the vertical load and the compression of the backfill at stage **m2** in test PS2 (Fig. 11), at which the loading pattern similar to the one at stage **a** was applied to the backfill of the pier after a slit had been made in the facing. Figure 12(c) shows the relationships between the vertical load amplitude and the compression amplitude of the backfill of the pier in each loading cycle of stages **a** and **m2**.

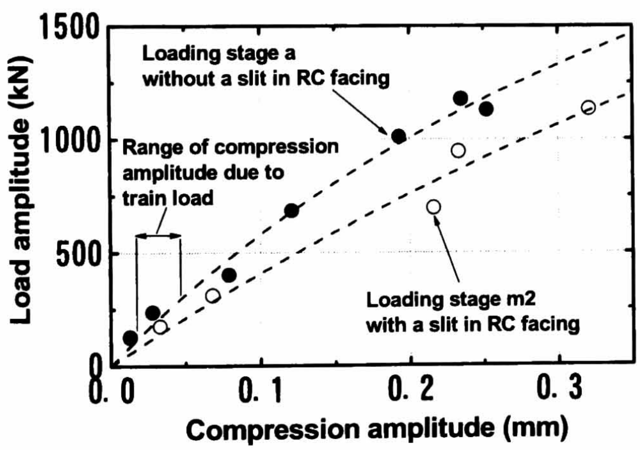
It may be seen that, in stage **a**, the peak-to-peak stiffness in each cycle decreased with an increase in the load amplitude, showing a noticeable non-linearity of the deformation characteristics of the backfill. In the same figure, the range of the measured transient compression of the backfill by train load, which ranged between around 0.02 and 0.04 mm, are shown (Uchimura et al., 2003). As the nominal weight of each coach was around 400 kN, it may be seen that the behaviour of the backfill during the post-service loading test PS1 that simulated train loads is not very different from the one observed when the pier was in service. However, it may also be



(a)

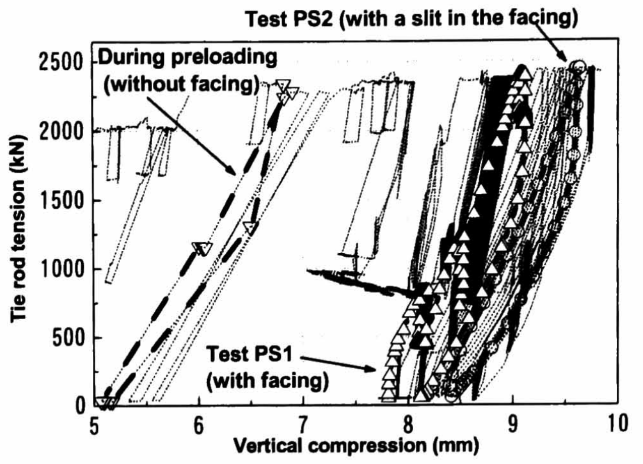


(b)

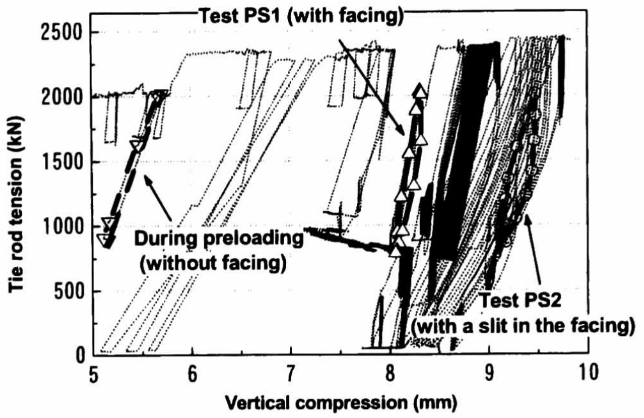


(c)

Fig. 12. Zoom-upped load-compression relationships during (a) stage a (Fig. 8), (b) stage m2 (Fig. 11) and (c) the relationships between the amplitude and the compression of each loading cycle



(a)



(b)

Fig. 13. Comparison of the stiffness of the pier between preloading stage and loading tests

noted that the stiffness during the cyclic loading test is slightly smaller than the one observed when the pier was in service. One of the reasons for this difference may be that the loading period in the loading test was about 8 to 10 minutes, which may have allowed some viscous deformation to take place. On the other hand, it is very likely that viscous deformation was significantly smaller in the deformation that took place by train loads, as the loading/unloading cycles were applied for only less than 10 seconds.

The secant stiffness obtained for each loading cycle of the loading states **a** and **m2** can be compared in Fig. 12(c). The pier showed slightly lower stiffness after making the slit in the facing. The possible reasons for this are:

a) a part of vertical load applied to the RC reaction block was supported by the facing without a slit in test PS1; and
b) the backfill had vertically swollen to some extent when left under fully unloaded conditions for four and a half months (stage **L**) between tests PS1 and PS2.

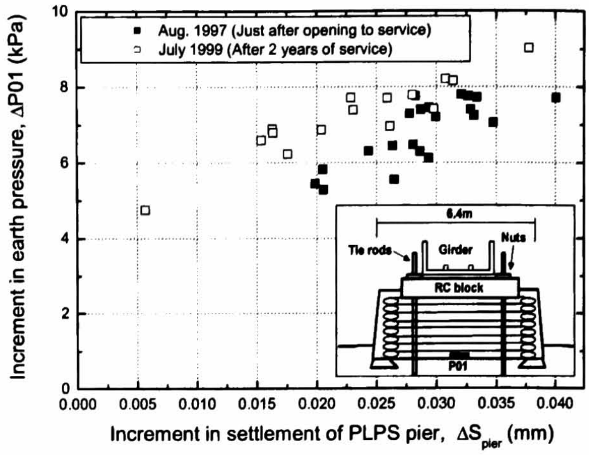
Based on the above, it is likely that a part of the train loads applied to the top RC block of the pier was supported by the facing. However, it was not larger than 30% of the total load applied to the top RC block.

It appears that the stiffness of the backfill of the pier slightly increased during a relatively long term of service. Figure 13(a) compares the behaviours of the backfill during a cyclic loading with an amplitude of 2,350 kN during the preloading stage (i.e., from stage 12 to stage 14, Figs. 4 and 5) with those during the two cyclic loading tests performed after the end of service (i.e., stage **i** in test PS1, Figs. 7 and 8; and stage **m1** in test PS2, Figs. 10 and 11). The amplitude of compression during cyclic loading at the preloading stage was equal to 1.8 mm, while those at stage **i** and stage **m1** were 1.0 mm and 1.2 mm, respectively. Similarly, Fig. 13(b) compares the behaviour during cyclic loading for a range of vertical load of 800–2,000 kN at the preloading stage (i.e., from stage 9 to stage 11, Figs. 4 and 5) with those during the loading test PS1 (i.e., stage **a**, Figs. 7 and 8 and Fig. 12(a) for details) and the loading test PS2 (i.e., stage **m2**, Figs. 10 and 11 and Fig. 12(b) for details). It may also be seen from Fig. 13(b) that the stiffness of the backfill had increased noticeably during the service period.

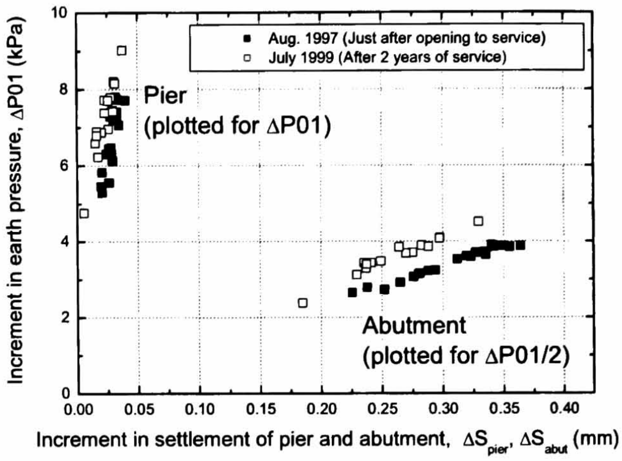
The possible reasons for such increase in the stiffness of the backfill during the long-term service could be due to:

1) a contribution of the RC facing that was cast-in-place around the backfill after the preloading procedure, as discussed above; and
2) some hardening effects of the backfill material due to the continuous static loading of prestress and the large number cyclic loading more than 10^5 times by train load during service.

With respect to these positive effects developed during service, Fig. 14(a) shows the relationships between the transient vertical earth pressure increment ΔP01 measured at the center of the bottom of the backfill and the transient vertical compression increment ΔS_{pier} of the backfill observed for each train passing immediately after opening to service (3 August, 1997) and after about two years of service (16 July, 1999). Uchimura et al. (2003) reported their detailed behaviours. The vertical compression was measured by displacement transducers attached to the top concrete block, which measures the relative displacement to the top end of rods vertically connected to the reference plates embedded at the bottom of the pier and the abutment. Despite this measurement system being very simple, it was successful in measuring the very small behaviours during several seconds. As the train load was not measured directly, these values of ΔP01 are only available information for the transient load acting on the backfill during train passing. It may be seen that the stiffness of the backfill became larger by around 20% after the first two years of service. The same trend of behaviour was also observed with the backfill of the abutment. That is, Fig. 14(b) shows the data for the backfill of the abutment A2 as well as those for the backfill of the pier presented in Fig. 14(a). For the abutment, ΔS_{abut} means the measured transient vertical compression increments, while the transient vertical earth pressure increments were estimated to be a half of the respective value of ΔP01 measured at the bottom of the backfill of the pier considering the structure of the bridge (Fig. 1). It may be seen that the stiffness of the backfill of the abutment also increased noticeably after the first two years of service.



(a)



(b)

Fig. 14. Stiffness of the pier and the abutment under traffic load: the earth pressure in the abutment is assumed to be a half of the one in the pier

Importance of Prestressing

With respect to the effects of preloading and prestressing on the stiffness of backfill, it may be seen from Fig. 14(b) that the stiffness of the backfill of the abutment during service was much lower than that of the preloaded and prestressed backfill of the pier. It may also be seen from Figs. 8 and 11 that the tangent modulus of the backfill of the pier noticeably increased with an increase in the instantaneous vertical load level. This trend in behaviour could be seen more clearly along the unloading curves than along the loading curves. That is, immediately after the start of unloading from the largest load 2,350 kN, the tangent stiffness of the backfill was extremely high, while the tangent stiffness decreased noticeably as the vertical load decreased towards zero. These results indicate that it is particularly important to maintain sufficiently high prestress, without unloading to very small or zero vertical load level, after the application of preloading, so that benefits by preloading are not largely lost. These trends in behaviour have also been confirmed by the loading tests on small-scaled models of reinforced soil pier in the laboratory (Shinoda et al., 2003a).

Importance of Preloading

The total vertical compression of the backfill of the pier (P1) was:

a) 7.5 mm at the end of the preloading stage (i.e., at stage 18, Figs. 3 and 5);
b) 8.0 mm at the end of service (i.e., at stage 19); and
c) 8.4 mm at the end of all the loading tests (i.e., at stage 22).

Despite that these values were measured at different load levels, the major part of the finally observed residual deformation of the backfill took place at the first preloading stage. In addition, most of the deformation during the preloading stage developed as creep deformations under constant loads (Origin to stage 1, Fig. 5). These facts indicate that, if preload had not been applied, the time-dependent deformation due to viscous properties would have been significant even with the backfill of the pier, which was highly compacted using well-graded gravel. The potential of such a large time-dependent deformation could be effectively suppressed by relevant preloading procedure applied to the backfill of the pier. This effect of preloading can be seen also from the following observations:

a) The backfill of the pier was continuously subjected to a high prestress of around 900 kN when subjected to a very large number of cycles of traffic load during the service period. Despite the above, the rate of residual deformation of the backfill during the service period was as small as 0.5 mm/year (from stage 18 to stage 19 in Figs. 3 and 5). It is very likely that this high performance would have not been achieved if the preloading and prestressing procedure had not been used.
b) In test PS1, the creep deformation rate of backfill became very high when the load approached the maximum preload level. That is, referring to Fig. 8, the creep deformation of the backfill was 0.2 mm at a vertical load of 2,350 kN, which is near to the maximum preload level, for 1 day at stage **c**, while it was only 0.03 mm at a vertical load of 2,000 kN for 14 hours at stage **b**.
c) The backfill expanded vertically at several creep loading stages during otherwise global unloading from higher load level. For example, the backfill vertically expanded and pushed up the top RC block, which was fixed to the tie rods, during the following creep loading stages under unloaded conditions; (1) between stage 2 and stage 3; (2) between stage 6 and stage 7; and (3) for a period after stage 16; (4) for other periods during the preloading procedure (Figs. 4 and 5); (5) at stages **d**, **f**, **h** and **j** in test PS1 (Figs. 7 and 8); and (6) stage **o2** in test PS2 (Fig. 15, a zoom-up of Fig. 11).

It should be noted however that the benefits of preloading described above could be largely lost once unloaded to a very low or zero load level. For example, referring to Fig. 15, despite the creep load was the same as stage **o2**, the pier exhibited again noticeable positive creep deformation at creep stage **o4** during otherwise global reloading from a very low load level.

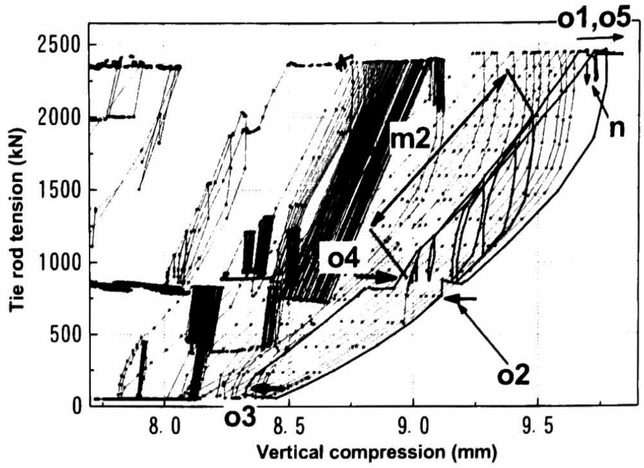


Fig. 15. Zoom-upped load-compression relationships during stage o (Fig. 11)

These trends of creep behaviour could be summarised as follows:

1) At creep loading stages during otherwise primary loading, significant compressive creep deformation tends to develop and the rate of creep deformation becomes larger at higher load levels.
2) At creep loading stages during otherwise global unloading, the creep deformation becomes significantly smaller or negative (i.e., expansion; the so-called creep recovery phenomenon).
3) At creep loading stages during otherwise global reloading after the backfill has been allowed to swell largely at a very low load level, the creep deformation could become noticeable again and could increase with an increase in the load level.

The preloading procedure is also effective to suppress the development of residual deformation due to cyclic loading. It is very likely that if the preloading and prestressing procedure had not been used, significant residual deformation would have taken place in the backfill of the pier by traffic load during service. This inference is supported by the following observations:

1) It may be seen from Figs. 7 and 8 that the total residual compression of the backfill due to 120 cycles of loading at stage **e3** in test PS1 was only 0.2 mm, despite the load amplitude applied to the backfill was relatively large, around 1,600 kN. This high performance could be a result of the preloading history and some residual compression developed for a long-term period of 4.5 years.
2) At stage **f1** in test PS1, 50 cycles of small cyclic load with amplitude of 400 kN were applied to the backfill of the pier during otherwise global unloading (*see* Fig. 16 for details). The backfill exhibited noticeable expansion during cyclic loading, likely reflecting the creep recovery phenomenon cited above. On the other hand, at stage **f2**, where 70 cycles of load with the same amplitude as stage **f1** was applied after allowing a swelling of 0.29 mm for 15 hours at a constant load between **f1** and **f2**, the backfill exhibit-

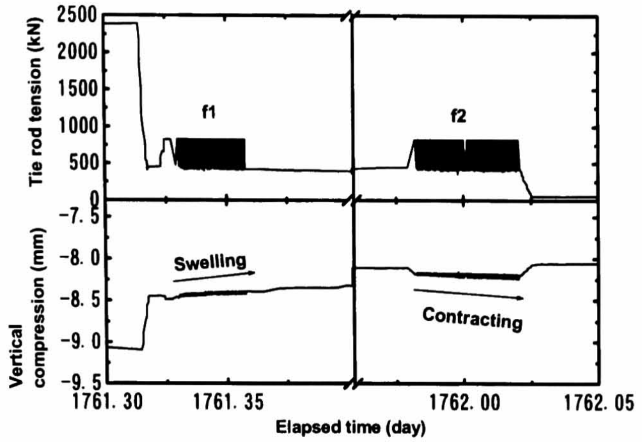


Fig. 16. Zoom-upped load-compression relationships during stage f1 and f2 (Fig. 7)

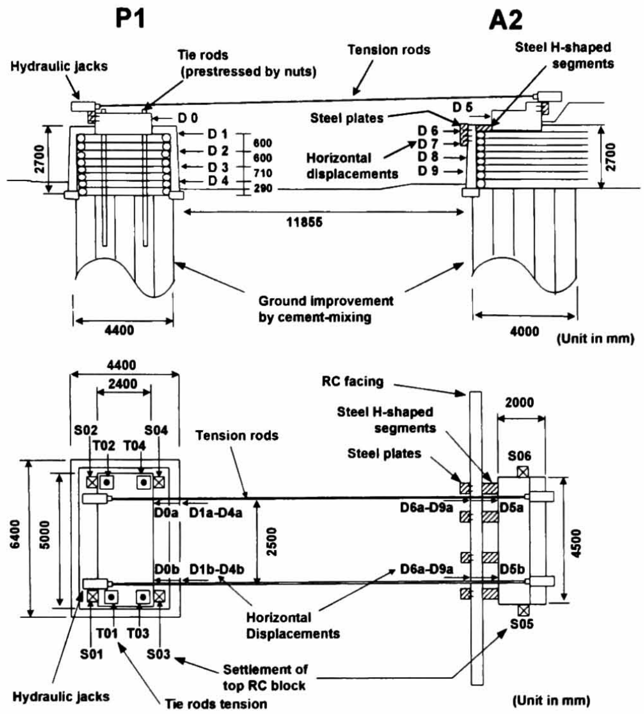


Fig. 17. Set up of horizontal loading tests performed in 2001

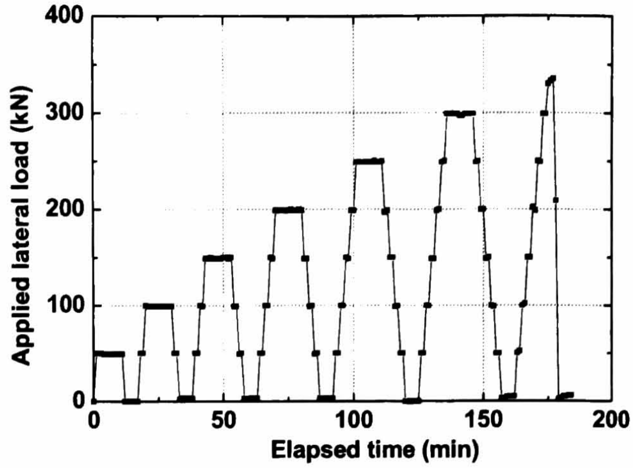


Fig. 18. Time history of applied horizontal load in the lateral loading tests of the pier and abutment

ed small but positive compressive residual deformation. This trend in behaviour is very similar to that of creep deformation.

The total residual strain of the backfill by cyclic loading at stage **m1** in test PS2 was relatively large, equal to 0.5 mm, as seen from Figs. 10 and 11, despite the number of cycles was only ten. This could be due to the following reasons:

1) A large amplitude of cyclic loads were applied between zero to the peak load;
2) The cyclic loads were applied after the backfill was allowed to swell at zero vertical load level for 4.5 months (stage **L**).
3) The cyclic loads were applied after a slit was made in the RC facing.

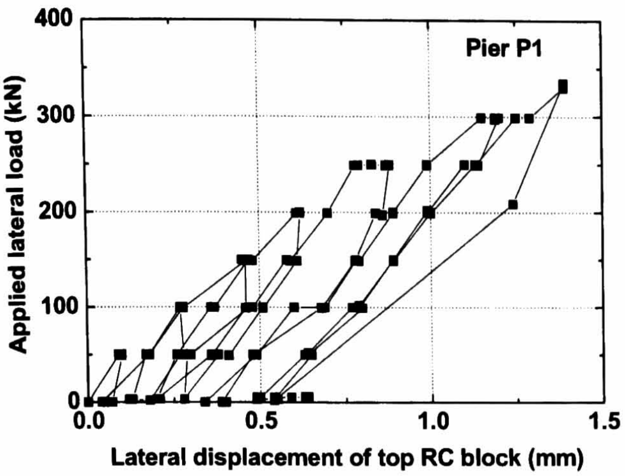
HORIZONTAL LOADING TESTS

Horizontal loading tests were performed on pier P1 as well as its companion reinforced soil structure, abutment P2, in June, 2001, just before the vertical loading test PS1. The top RC blocks of the pier and the abutment, on which the bridge girder was seated, were connected to each other by using two steel tension rods (Fig. 17). A pair of hydraulic jacks was arranged behind the top RC block of the pier to apply pulling force. Therefore, the same horizontal load was applied to the top RC blocks of the two structures instantaneously. Steel H-shaped segments were installed between the top RC block and the RC facing in order to increase the resistance of the top RC block against horizontal load, because it was not anchored to the backfill. The top part of the RC wall facing of the abutment, where the H-shaped segments were attached, was reinforced by bolting steel plates on it to prevent local failure. On the other hand, the top RC block of pier P1 was separated from the RC facings by installing a soft material around the top RC block as seen in Fig. 6 so that nearly all the horizontal load applied to the top RC block was transmitted to the backfill.

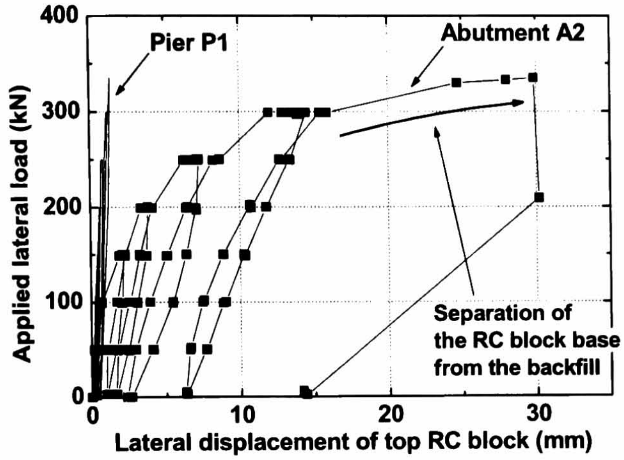
The jack force and the horizontal displacements of the top RC blocks and the facings were measured as shown in Fig. 17. The instrumentations that had been used from the start of this research project, including the gauges to measure the compression of the backfill between the top RC block and the bottom of the backfill and the tie rods tension, were also used in the horizontal loading test.

Figure 18 shows the time history of applied horizontal load. Seven cycles of loading and subsequent full unloading were applied while their peak load level was increased by 50 kN step by step up to 350 kN. These procedures simulates cyclic seismic *horizontal load*. Each peak load was maintained for ten minutes except the last cycle in which the abutment failed.

Figure 19 shows the relationships between the horizontal load and the horizontal displacement at the top RC blocks of the pier and abutment. The maximum displace-



(a)



(b)

Fig. 19. Relationships between applied lateral load and lateral displacement of the top RC blocks: (a) pier P1 and (b) abutment A2

ment of the top RC block of the pier was only 1.4 mm at a horizontal load of 350 kN, and nearly half of it was recovered by unloading. At this stage, the base of the top RC block of the abutment started largely heaving losing a contact with the backfill, as shown in Fig. 20, and therefore a larger horizontal load was not possible to apply. The horizontal displacement of the top RC block of the abutment was as large as about 30 mm, which was about 21 times larger than the pier, at a horizontal load of 350 kN, the horizontal displacement of the top RC block of the abutment was about 30 mm. It is very likely that if steel H-shaped segments had not been installed between the top RC block and the RC facing of the abutment, the top RC block had lost stability at a much lower horizontal load. This large difference in the structural performance against horizontal load between the pier and abutment is no doubt due to the top RC block of the pier not being connected firmly to the backfill by prestressing using four tie rods.

Figure 21 shows the horizontal displacements at several heights of the facing at different horizontal load levels.

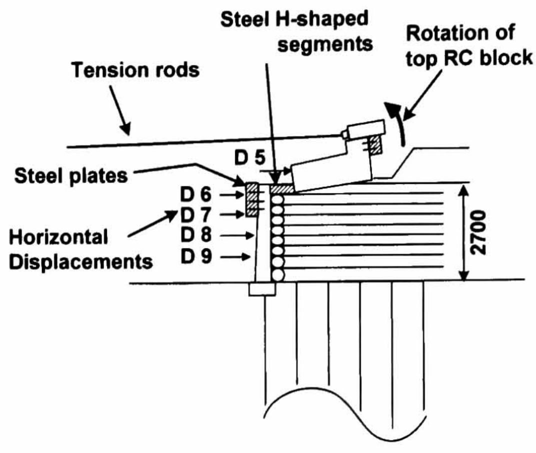
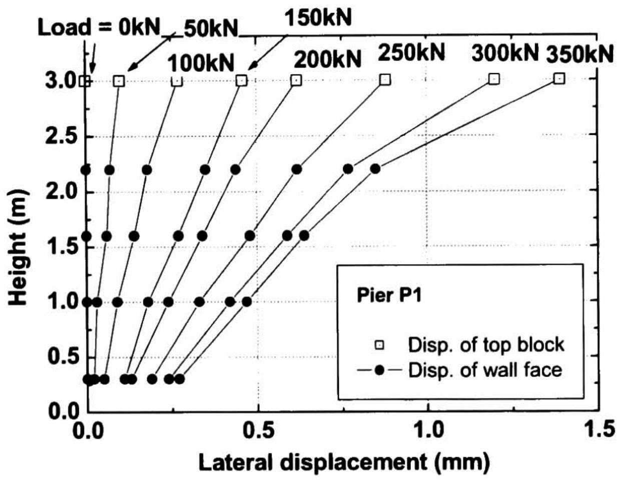
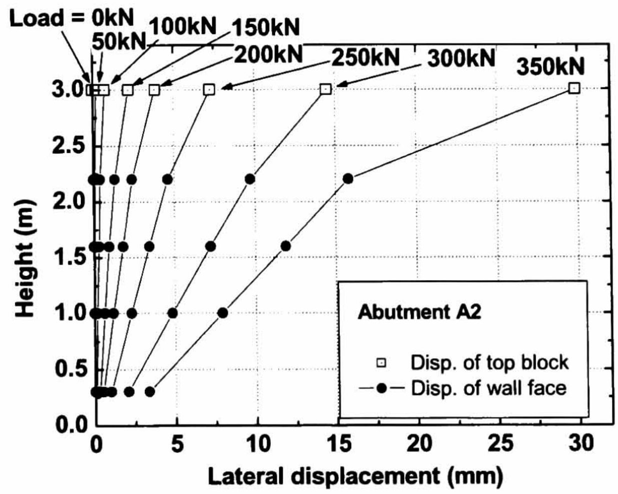


Fig. 20. Schematic diagram of the movement of the top RC block on the abutment due to horizontal loading



(a)



(b)

Fig. 21. Lateral displacements of the RC facings and the top RC blocks: (a) pier P1 and (b) abutment A2

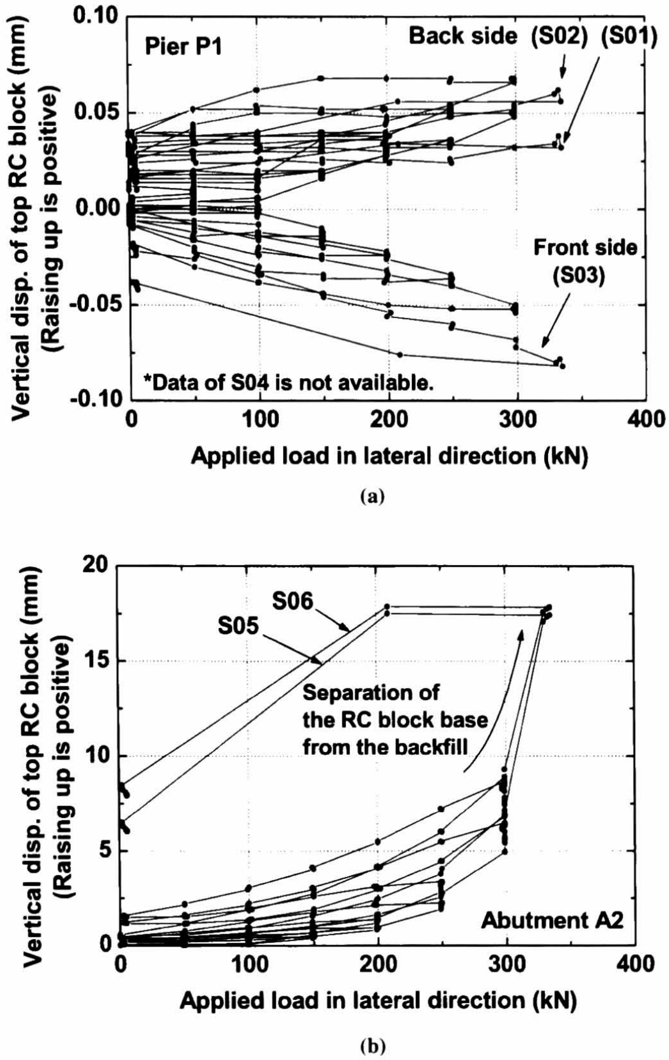


Fig. 22. Relationships between the applied horizontal load and the vertical displacement of the top RC blocks of (a) pier P1 and (b) abutment A2

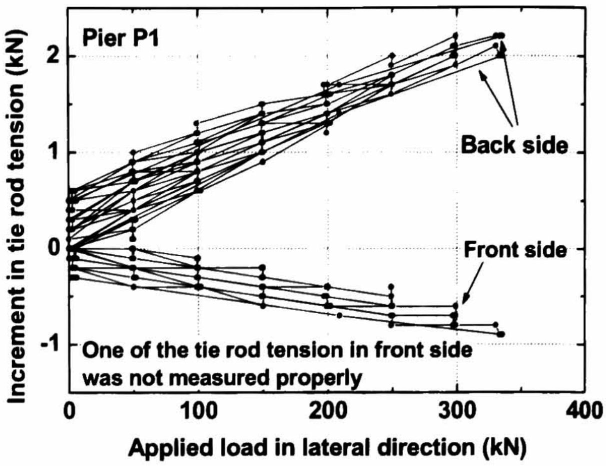


Fig. 23. Relationships between the applied horizontal load and the increment in the tie rod tension of pier P1

The displacements of the top RC blocks at the height of 3 m are also plotted together. The following trends in behaviour may be seen from these figures:

1) The horizontal displacements of the facing of the pier are substantially smaller than the facing of the abutment, confirming the results presented in Fig. 19.
2) The center of rotation of both facings of the pier and abutment are below the base of facing.
3) The horizontal displacement of the top RC block of the pier relative to the facing (thus the backfill) was not negligible, but it was much smaller than that of the abutment, especially at the higher load level.

Figure 22(a) shows the vertical displacements at the back and front sides of the top RC block of the pier measured with displacement transducers S01, S02 and S03 (*see* Fig. 17). Unfortunately, gauge S04 malfunctioned during this test. Here, "front side" means the side of the RC block facing nearer to the abutment. It may be seen from this figure that, the front side of the top RC block of the pier settled down while the back side heaved up by nearly the same amount due to the horizontal loading. Correspondingly, the tension in the tie rods on the front side decreased while that of the back side increased (Fig. 23). These behaviours indicate that the pier behaved as a monolith exhibiting bending deformations when subjected to horizontal loads at the top RC block. Similar bending deformation would take place due to earthquakes.

It is important to note that such a bending deformation of the pier was restrained by the tie rods in the following two ways:

1) Part of the applied moment was supported by the tie rods.
2) The resistance of the backfill against bending moment became sufficiently large due to the high stiffness of the backfill obtained by a high prestress.

Such functions of the tie rods as described above to restrain the bending deformation of reinforced soil structure have been observed in static cyclic horizontal loading tests (Uchimura et al., 2001) and shaking tests (Shinoda et al., 2003b); both were performed on small-scaled models of PLPS geogrid-reinforced soil pier.

A high prestress is also important to attain a high shear resistance at the interface between the top RC block and the crest of the backfill as well as a high stiffness against shear deformation of the backfill. The maximum horizontal load applied in the tests was 350 kN, which is 30% of the total weight of the pier. Therefore, this horizontal loading test somehow simulated a middle class horizontal seismic load. The prestress at the horizontal loading test was around 800 kN, which was more than twice of the maximum applied horizontal load. Further, the self weight of the top RC block is 240 kN. Therefore, the mobilised friction angle at the base of the RC block is estimated to be arctan {350 kN/(800 kN + 240 kN)} = 19 degrees. The mobilized friction angle in the backfill should be somehow smaller because the vertical confining pressure is larger by the self weight of the backfill. On the other hand, the peak angle of internal friction of the backfill material obtained from triaxial tests on specimens compacted to dry density of 1.95 g/cm^3 was 60 degrees (Uchimura et al., 2003). As the dry density of the pier backfill was 1.91 to 2.17 g/cm^3, which is only slightly higher than that of the triaxial specimen, it is reasonable to estimate that the friction angle to be around 60 degrees at the interface between the RC block and the backfill and in the backfill. It can therefore be seen that the estimated mobilised angle of friction at the interface and in the backfill were much smaller than their peak values. This analysis indicates that the safety factors for sliding failure at the interface between the top RC block and the backfill and the shear failure along horizontal planes in the backfill were large enough. It is to be noted that these safety factors for a given horizontal load could be increased more if the prestress is higher.

On the other hand, Fig. 22(b) shows the vertical displacement at the center of the top RC block of the abutment measured at S05 and S06 (Fig. 17). A sudden increase in the vertical displacement means a loss of contact between the top RC block and the backfill, which resulted in a loss of the shear resistance against the horizontal load.

CONCLUSIONS

This paper reports the behaviours of a PLPS geogrid-reinforced soil pier for a railway bridge observed in vertical and horizontal loading tests. They are compared with the behaviours of the pier for about 5 years including the service period as well as those at the preloading stage in construction. They are also compared with those of a geogrid-reinforced soil abutment supporting the same bridge, which was neither preloaded nor prestressed. The following conclusions could be derived from the above:

1) The backfill of the pier showed very high stiffness in the vertical loading test, which is consistent with the high performance observed during service.
2) It appears that the backfill was hardened to some extent by ageing effect or strain hardening effect or both due to continuous prestressing and a large number of cycles of traffic loads applied for a long service period.
3) The ratio of the load supported by the RC facing cast-in-place around the backfill to the total applied load during service is estimated to be 30% or less. So, a high performance of the pier during service could be mostly attributed to the preloading and prestressing procedure on the backfill.
4) Despite that the backfill of the pier consisted of well-graded and well-compacted gravel, significant creep deformation due to sustained loading and residual deformation due to cyclic loading were observed. However, these deformations decreased substantially by preloading. The backfill exhibited noticeable time-dependent expansions at sustained loading stages under largely unloaded conditions. On the other hand, the time-dependent compression increased again at the reloading stage after the backfill had been fully unloaded to zero level and allowed to swell largely. These trends in behaviour confirmed the importance of preloading and prestressing to maintain a high stiffness of the reinforced soil structures and minimize their residual deformation.
5) When subjected to large horizontal load at the crest of the structure, the PLPS geogrid-reinforced soil pier behaved like a monolith showing a very high resistance against the over-turning moment and the shear load. The tie rods contributed sufficiently to the high performance by directly supporting the major part of the applied load and indirectly prestressing the backfill. In contrast, the stability of the top RC block placed on the crest of the backfill of the abutment against horizontal load was much lower, because the block was not connected to the backfill. These trends in behaviour indicate that the preloading and prestressing is also very effective in achieving high seismic performance of geogrid-reinforced soil structures.

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