Long-term behaviour of preloaded and prestressed geogrid reinforced soil bridge pier

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ABSTRACT: The performance for nearly four years of the first prototype bridge pier of geogrid-reinforced gravel is described. The pier was constructed for a railway in the summer of 1996 in Fukuoka City, Kyushu, Japan to support two 16.5 m-long simple-supported girders for a single-track railway. The backfill was a well-graded gravel of crushed sandstone reinforced with geogrid. The pier was made very stiff by applying a vertical preload allowing large creep deformation to occur in advance, and by keeping a high prestress for the service period of the pier. The pier exhibited a transient compression of only 0.02 to 0.03 mm by train load. Because of the above, the pier has exhibited very small relaxation of tie rod tension and residual settlement. A series of loading tests on a scaled model was performed in the laboratory to evaluate the effects of preload and prestress on the transient and residual compression of the reinforced backfill under cyclic vertical load simulating traffic load acting on the crest of the backfill. It was found that the prestress should be large enough to make the elastic stiffness of backfill sufficiently high, and the preload should be sufficiently larger than the sum of prestress and live load.

1 INTRODUCTIONS

A new construction method, the preloaded and prestressed (PLPS) reinforced soil method, has been proposed; a reinforced backfill is made very stiff against vertical load by vertical preloading and prestressing (Tatsuoka et al. 1997a.). The mechanism of the PLPS method was demonstrated by tests on a full-scale model embankment and creep-relaxation tests on large triaxial specimens of the backfill gravel (Uchimura et al. 1996).

This paper reports the construction and behaviour of the first prototype PLPS geogrid-reinforced soil (GRS) bridge pier. The bridge pier was constructed to support a pair of temporary railway girders, each 16.5 m in length, in the summer of 1996 in Fukuoka City, Japan (P1 in Fig.1). This pier has been opened to service since the summer of 1997. The behaviours of the PLPS pier and an ordinary-type geogrid-reinforced soil abutment for the bridge (A2 in Fig. 1), which was not preloaded nor prestressed, when subjected to a large number of cyclic traffic load, were compared. The comparison showed very clear and significant advantages of the PLPS method.

Cyclic vertical loading tests on small-scale models of PLPS pier were performed to evaluate the effects of preloading (PL) and prestressing (PS) on the behaviour of the pier under cyclic load. The results showed that it is possible to restrain transient and residual deformation of reinforced soil structure to negligibly small values if sufficiently large preload and an appropriate level of prestress are applied.

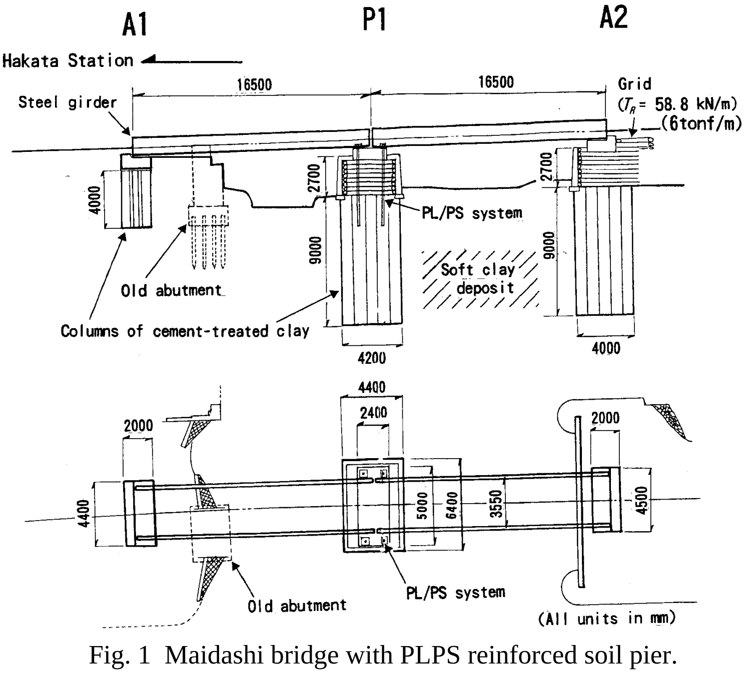

Fig. 1 Maidashi bridge with PLPS reinforced soil pier.

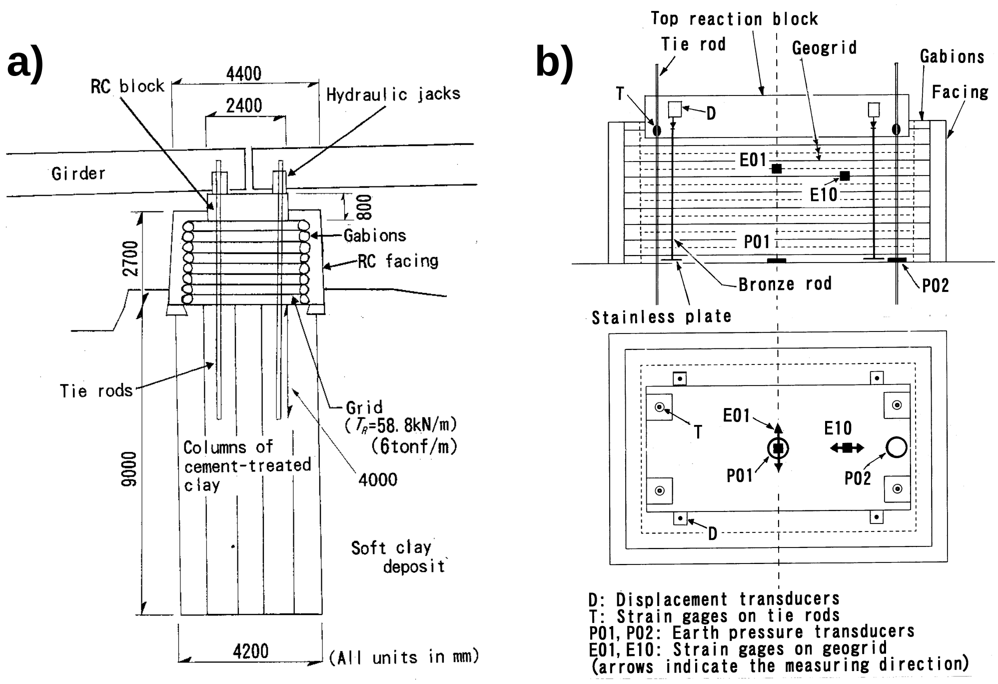

Fig. 2 a) Details of the pier, and b) arrangement of the instrumentation described in this paper.

2 BEHAVIOUR OF PROTOTYPE PLPS BRIDGE PIER

2.1 *Construction*

The PLPS GRS bridge pier is 6.4 m x 4.4 m in cross-section, and 2.7 m in height (Fig. 2a). The design dead load by the girder weight and live load by train including impact load are 196 kN and 1,280 kN, respectively. Before constructing the pier, a subsoil layer of an about 9 m-thick very soft clay was improved by producing in-situ 0.8 m-in-diameter cement-mixed soil columns. A 1 m-thick surface clay layer of the pier was improved for the whole cross-section by cement-mixing to support vertical compression to the backfill. The lower ends of four steel tie rods were anchored into the cement-mixed soil columns for a length of 4 m. The nominal yield tensile force of the tie rod is 1,034 kN. Then the backfill was constructed with a help of gravel-filled bags stacked along the periphery of each gravel layer, while the bags were wrapped around with the reinforcement. A well graded gravel of crushed sandstone (D_{max} = 30 mm, D_{50} = 0.9 mm, U_c = 16.5, ϕ = 60° by triaxial compression tests) was used for the backfill. A geogrid reinforcement of polyvinyl alcohol coated with polyvinyl chloride (PVC) was used, whose nominal rupture strength is 73.5 kN/m and the nominal stiffness is 1,050 kN/m at strains less than 1 percent. The reinforcement was arranged in the same way as for a GRS-RW with one facing at the end under plane strain condition with the same height as the actual pier, which rendered a vertical spacing of the reinforcement of 30 cm. As the pier has a rectangular prismatic shape with two pairs of wall faces in orthogonal directions to each other, each cross-section having one pair of wall faces were designed independently. By overlapping the two cross-sections, the actual average vertical spacing of the reinforcement became 15 cm.

The construction of the backfill took five days by a team of five workers. Preloading started ten days after casting the top reaction RC block (5 m-long, 2.4 m-wide and 0.8 m-high). A vertical preload of 2400 kN (an average vertical pressure of 200 kPa) was applied to the backfill of the pier through the top reaction block by using four hydraulic jacks installed at the top of the tie rods. The backfill was compressed by 8 mm during preloading. Then the load was reduced to 970 kN, and the top ends of the tie rods were fixed to the top RC block to maintain the compressive stress (i.e. prestress) in the backfill. Finally, full-height rigid facings were cast-in-place around the backfill. The total construction period was about 1.5 months. The total construction cost for the pier was estimated to be about a half of that for an equivalent conventional RC pier supported by a pile foundation.

The pier P1 was instrumented as shown in Fig. 2b. Tension in each tie rods was measured by using strain gages attached to it. Vertical compression of the backfill was measured at four points on the top reaction block as settlement relative to stainless plates placed at the bottom of the backfill. Grid tensile strains were measured by using strain gages at 32 points in total. A half of them are in the railway direction and the rest are normal to it.

The abutment A2 was constructed using the same method and materials as the pier, except that it has only one GRS retaining wall having reinforcement with a vertical spacing of 30 cm, without PL and PS, and the both sides are exposed slopes (1.5:1.0 in H:V) without a facing.

2.2 *Observed behaviour*

The vertical compression and the tie rod tension of the pier have been observed for about 4 years after construction (Fig. 3a). In addition, the compression of the ordinary type reinforced soil abutment with full height rigid concrete facing (A2 in Fig. 1), which was not preloaded nor prestressed, has been also observed. The PLPS pier was compressed by about 8 mm for the first 10 days of preloading period, and subsequently it was compressed very slowly under the prestressed condition. Even after having opened to service, the compression rate of the pier was not changed, kept to a very small value. Corresponding to the compression, the tie rod tension decreased only very gradually. These rates of change are small enough for a temporary use for about 5 years. On the other hand, the abutment, without PL and PS, was compressed by about 3 mm due to its own weihgt and the weight of the girder for 10 months after construction, and very quickly after opening to service. The compression is still continuing after more than 3 years. These comparisons show that the preloading and prestressing procedure is very effective to reduce vertical compression of the reinforced backfill by more than 10^5 cycles of traffic load, applied for a long period.

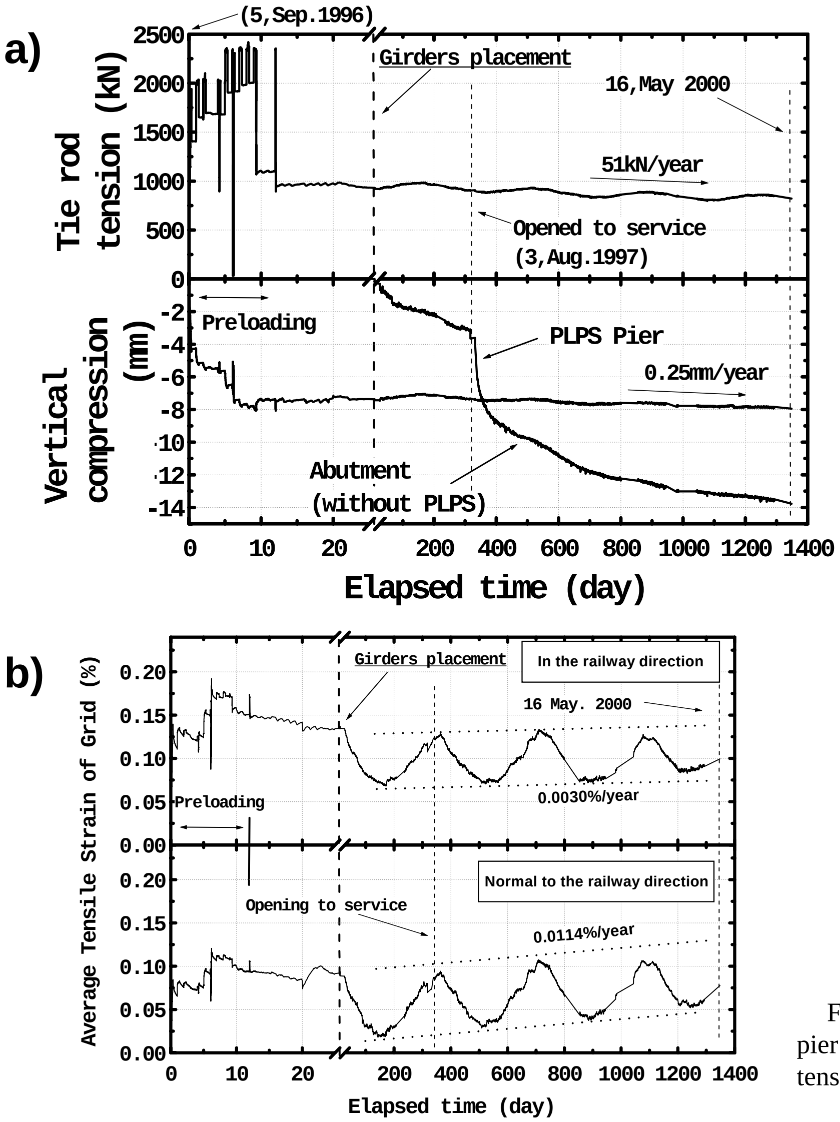

Fig.3 Long-term behaviours of a) the pier and abutment and b) average of grid tensile strain of the pier in each horizontal

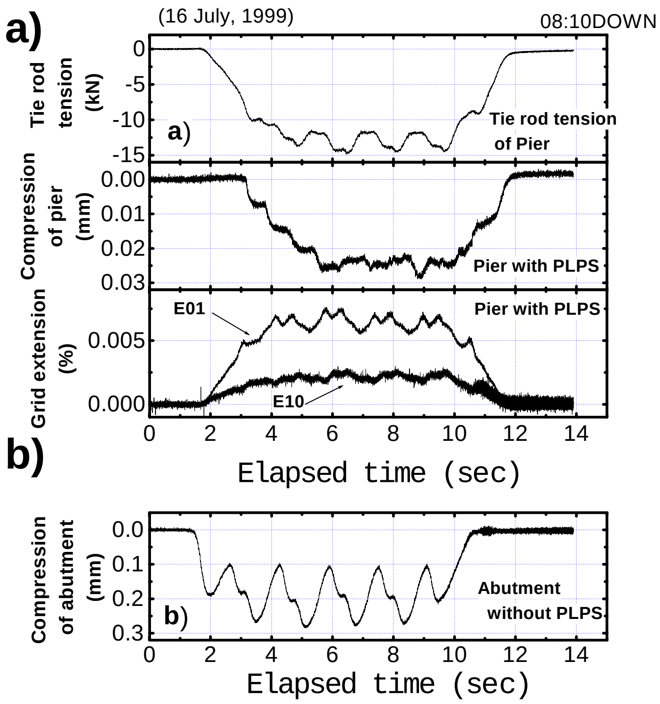

Fig. 4 Behaviours of a) the pier and b) the abutment during a train passing after 2years of service (July, 1999)

Fig. 3b shows the grid tensile strains in the parallel and normal directions to the railway axis respectively. Some annual change is observed, which is probably due to temperature effects on the measurement system. Although the general trend of the deformation of grid is extending, its rate is very small enough for a long-term use.

Fig. 4 shows the behaviour of the tie rod tension and vertical compression of the PLPS pier when a train passed over the bridge 2 years after opening to service (July, 1999). The train had six carriages, each weighing about 500 kN. The pier backfill was compressed essentially elastically by 0.025 mm, which corresponded to a soil strain of 0.001%, while the tie rod tension temporarily decreased by 15 kN. The grid strains also responded to the load, but thier changes were very small and nearly elastic. At the same time, the abutment, without PL and PS, was compressed by 0.3 mm, which was about ten times as large as that of the PLPS piers. The same kind of measurement was also performed just after opening to service in August of 1997 (Uchimura et.al. 1998). The observed behaviour was nearly the same as that in 1999. This means that the properties of the pier and abutments had not changed essentially during those 2years in service.

It can be understood that the strain amplitude of the stiffened PLPS pier was so small that the backfill deformed nearly elastically, while that of the abutment, without PL and PS, was significantly larger, causing plastic compression. This could be the major cause for the large differences in the long-term compression between the pier and the abutment mentioned here.

2.3 *Discussion*

The comparison of the long-term behaviours of the pier and the abutment showed a high stability of GRS PLPS structures. It is also shown that the preloading and prestressing procedure has significant effects to reduce vertical compression of the pier backfill under both long-term dead load and a very large number of cyclic traffic load. Their mechanism could be summarized as follows:

1) The backfill can be effectively compacted and significantly large preload can be applied without damaging the backfill because it is reinforced.
2) The backfill is very stiff, exhibiting nearly elastic behaviour, because large preload has been applied and then released. In addition, the potential of creep deformation and relaxation of the backfill soil has been substantially decreased by the preloading procedure.
3) The backfill is always under a high compressive stress conditions by remaining prestress, making its elastic stiffness very high.
4) Large part of the tensile strains in the reinforcements induced by preloading remains after the vertical load is decreased to the prestress level. Therefore, the reinforcement confines the backfill more efficiently than in the case without preloading.
5) The load working on the top of the backfill is always in equilibrium with the sum of the external load applied on the top reaction block and the tie rod tension. Therefore, when external compressive load is applied, the reduction in the tie rod tension associated with vertical compression of the backfill results in a reduction in the stress increment on the backfill. This mechanism decreases the compression of the backfill.

3 MODEL TESTS OF PLPS PIER

3.1 *Method*

A number of loading tests on scaled rectangular prismatic model of GRS pier was performed to evaluate the roles of preloading and prestressing in restraining vertical compres-

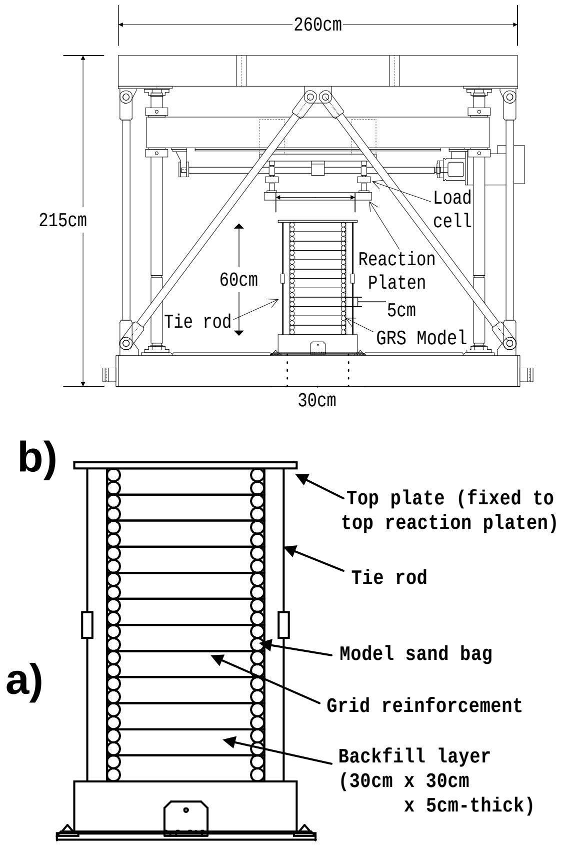

Fig. 5. a) Loading frame and b) GRS pier model.

sion of the PLPS structures. Fig. 5 shows the model (60 cm high and 30 cm×30 cm in cross-section) and loading system. Each model consists of 12 layers of 5 cm-thick sub-layers of dense air-dried Toyoura sand (D_{50} = 0.2 mm, U_c = 1.46 and e_{max} = 0.977 and e_{min} = 0.597), reinforced with model grid layers with a vertical spacing of 5 cm. The backfill part was produced by pluviating sand particles through air from multiple sieves to a void ratio of 0.63 (D_r = 91%).

The periphery of each sub-sand layer was protected with model sand bags, which were about 3.5 cm in diameter, wrapped-around with the model reinforcement grid. The model grid was made of polyvinylalcohol, having an aperture of 20 mm. The thickness and width of grid member were 1 mm and 2 mm respectively. The stiffness at a tensile strain of 5 % and the rupture strength from tensile tests at a strain rate of 5 %/min were

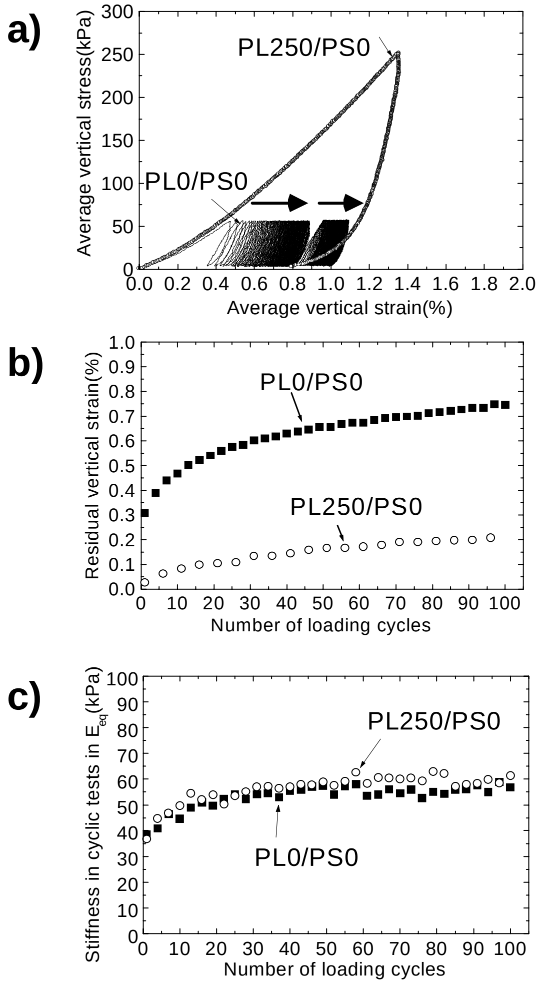

Fig. 6. Effects of preloading

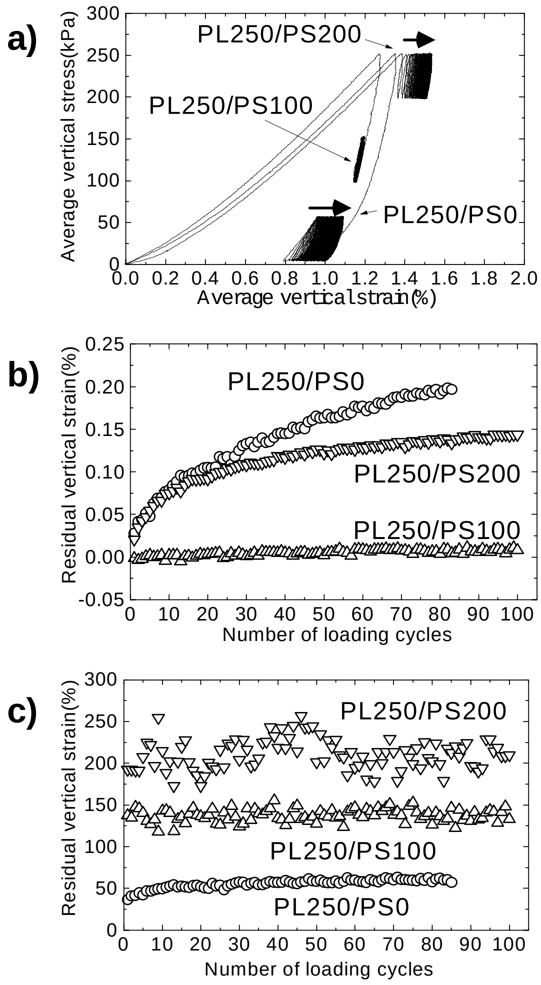

Fig. 7. Effects of prestressing

196 kN/m and 30.4 kN/m respectively.

A top reaction block (a steel platen of 5 cm thick and 45 cm × 45 cm in cross-section with a weight of 282 N) was placed on the top of the completed reinforced backfill. Fig. 5a shows the stage before placing the top reaction platen on the model. Vertical load was applied on the top reaction block using four double-action air cylinders.

The level of preload and prestress was controlled. Herein, for example, a notation "PL250/PS100" means a loading pattern in which axial load of 250 kPa was applied as the preload, and it was unloaded to 100 kPa as the prestress, and then cyclic load with 50 kPa of double amplitude was applied for 100 times. Many tests were performed without using tie rods, that is, the compressive load was applied and controlled by means of air cylinder on the top reaction block to simulate the case where the tie rod tension is kept constant.

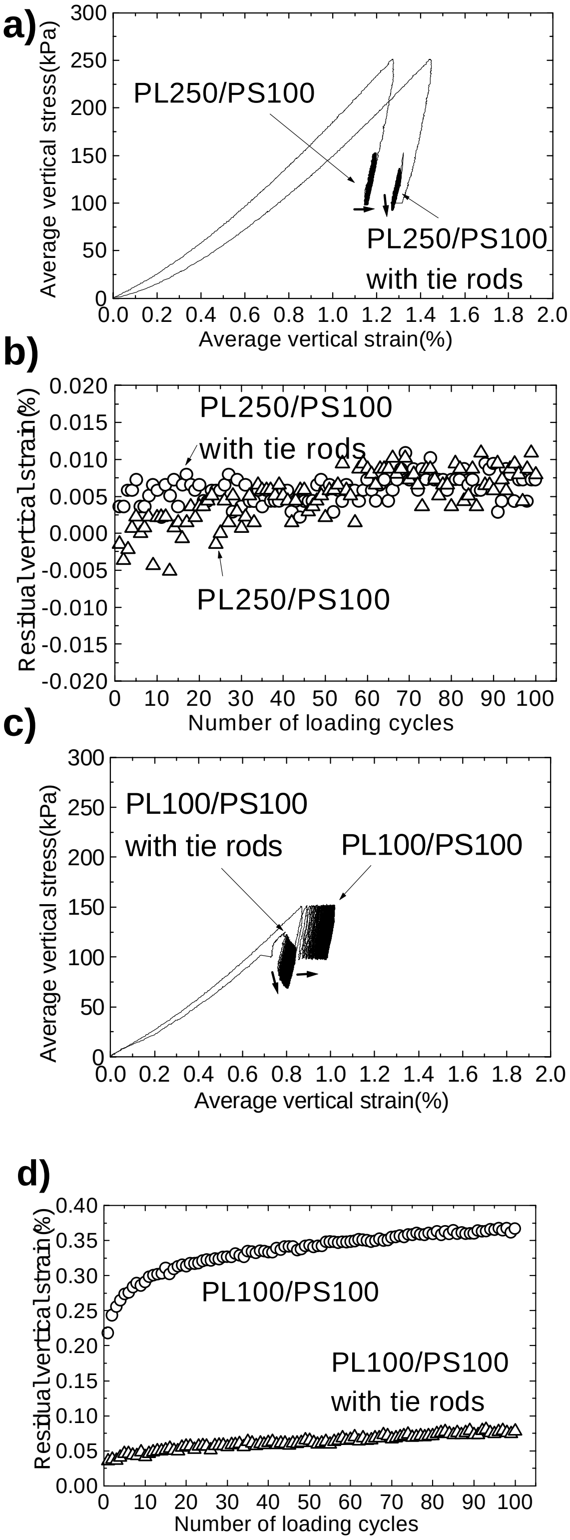

Fig. 8. Test results with and without tie rods

In the several tests with tie rods, prestress was maintained by means of four steel wires as tie rods fixed to the top and bottom reaction steel platens. To introduce tension as prestress in the tie rods, axial load was first applied by using the air cylinders to the top reaction platen, which was then gradually decreased being replaced by tie rod tension introduced manually.

3.3 *Effect of preloading and prestressing*

Fig. 6a compares the relationships between the average axial stress on the top of the backfill and the average strain of the backfill obtained from two tests, PL250/PS0 (i.e. preloaded to 250 kPa and then totally unloaded) and PL0/PS0 (no preloading nor prestressing) without tie rods. The cyclic residual strain is defined as the amount of accumulated strain after starting axial cyclic loading. The cyclic residual strain decreased substantially by preloading (Fig. 6b), probably because preloading had developed large irreversible strains in advance. In test PL250/PS0, the model exhibited a large swelling to an total axial strain of 0.8 % when unloaded from the high preload level (250kPa) to zero pressure (Fig. 6a), which should have largely softened the model. This large swelling was caused by release of elastic energy in the backfill and reinforcement stored by preloading. The effects of preloading on the equivalent Young's modulus E_{eq}, defined as the ratio between the amplitudes of stress and strain in each cycle, are insignificant.

Fig. 7a compares the behaviours when cyclically loaded at three different prestress levels (PS = 0,100 and 200 kPa) with the same preload level of 250 kPa without tie rods. The residual strain in this test was smallest in the case of PL250/PS100, i.e. PS is nearly half of PL (Fig. 7b). The different behaviours among the three tests should be explained by the following two factors:

1) swelling in test PL250/PS0 largely destabilized the sand structure that has been developed by preloading; and
2) the peak stress state during cyclic loading was the same with a preload of 250 kPa in test PL250/PS200, resulting into development of large irreversible strain by cyclic loading.

On the other hand, the E_{eq} values became larger with a higher prestress (Fig. 7c). This is because the E_{eq} value for a given strain amplitude is a rather unique function of the instantaneous axial stress (e.g., Tatsuoka et al. 1997b).

3.4 *Effects of using tie rods*

In the prototype pier, preload was applied by using hydraulic jacks attached to the top ends of the tie rods, and the tie rod tension was reduced to about a half of the preload. Then the tie rods were fixed to the top reaction block to bring the structure to the prestressed condition, under which the tie rod tension reduces when the backfill exhibits axial compression. Fig. 8a and b compare the behaviours of models with and without tie rods for the case of PL250/PS100. This case of PL250/PS100 with tie rods best simulates the prototype among those performed in the present study. The residual vertical strain were similar and acceptable against cyclic loading for both cases with and without tie rods. On the other hand, in the cases the maximum preloading was the same level as prestress, PL100/PS100 for example, the residual strain became substantially smaller by using tie rods (Figs. 8c and d). This is because the axial stress in the backfill decreases with the residual compressive strain in the cases with tie rods, gradually bringing the backfill to unloaded conditions.

4 CONCLUSIONS

As shown by the long-term behaviour of the prototype PLPS reinforced soil bridge pier, preloading and prestressing can restrain the vertical compression of the backfill very effectively if appropriately designed. The model test results showed that the residual deformation by cyclic load becomes smaller by applying larger preload. However, the residual deformation becomes unexpectedly large if the preload has been fully unloaded, caused by large plastic swelling induced by release of elastic energy stored in soil and reinforcement. If an appropriate level of prestressing is remained, the backfill becomes very stiff, as the stiffness increases as the current axial stress increases. Moreover, if the tie rods are used, their tension reduces with compression of the backfill resulting in reduction in the total load on the backfill. By these factors, the total residual strain becomes very small for properly designed PLPS GRS structures.

Based on the above, it is recommended that the level of preload and prestress is designed as follows:

1) the backfill is vertically preloaded to a load level of more than two times of the total design vertical load including dead and live load.

2) the vertical load is decreased to a prestress level of about a half of the preload level, so that the maximum load during service does not exceed the preload.

It should be finally underlined that sufficiently large preload and prestress could not be applied to usual unreinforced backfill without excessive deformation or failure. This method is possible only for well-reinforced well-compacted backfill.

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