

INCREASING THE STIFFNESS OF MECHANICALLY REINFORCED BACKFILL BY PRELOADING AND PRESTRESSING

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ABSTRACT

A new construction procedure applying a large preload to a mechanically reinforced backfill structure and keeping a large prestress while the structure is in service is described. This construction procedure was developed to substantially decrease transient and residual compression of reinforced backfill by long-term vertical cyclic loading, such as traffic load. A very high performance for about four and a half years of a prototype railway bridge pier of preloaded and prestressed geogrid-reinforced gravel backfill, which validated the advantages of this construction procedure, is described. A series of model loading tests performed in the laboratory to find the relevant preloading and prestressing procedure are described. It is shown that the transient and residual compression of reinforced backfill by vertical cyclic loading can be made very small by preloading the reinforced backfill and then decreasing the load level to a prestress level that is about a half of the preload level, while keeping the difference between the preload and prestress levels sufficiently larger than the amplitude of subsequently applied cyclic load. Effects of different combinations of backfill soil type and reinforcement material, the use of tie rods and the application of pre-cyclic preloading are reported. Negative effects of the swelling of backfill during unloading are discussed.

Key words: cyclic loading, model test, preloading and prestressing, reinforced backfill, Young's modulus (IGC: E14/K14)

INTRODUCTION

Geogrid-reinforced soil (GRS) retaining walls (RWs) are now widely used, mainly because of their high cost-effectiveness as well as their satisfactorily high performance (e.g., Tatsuoka et al., 1997a). This result can be attributed to the fundamental advantage of reinforced soil RWs in the stability mechanism over conventional type RWs (including gravity type and cantilever RWs). That is, the conventional type RWs are basically a cantilever structure. Therefore, lateral sliding load and overturning moment working at the bottom of a RW structure and the corresponding values working inside the wall structure are basically proportional to, respectively, H^2 and H^3 , where H is the wall height. For this reason, conventional type RWs become very massive and expensive with the increase in the wall height, in particular when the wall height exceeds, say, 10 m. This trend becomes more significant when the RW is designed considering seismic force. In addition, when a conventional type RW is constructed directly on the ground, the stability of the RW could be controlled by the bearing capacity of the supporting ground immediately below the RW. Therefore, unless the supporting ground is stiff enough, the displace-

ment of RW upon the construction of backfill could exceed the specified allowable value, which is usually very small with reinforced concrete (RC) structures. For these reasons, a conventional type RW is usually supported by a deep foundation, such as a pile foundation, unless the ground is very stiff. This second factor also tends to increase the cost of RW.

In contrast, the stability of reinforced soil RWs depends only partly on the bearing capacity of the supporting ground below the facing and the backfill zone immediately behind the facing, while the allowable deformation of the wall during construction is usually much larger. Therefore, reinforced soil RWs, including GRS RWs having a full-height rigid (FHR) facing, are usually not supported by a pile foundation. To alleviate possible damage to the connection between a FHR facing and reinforcement layers by relative settlements between the facing and the backfill and to control precisely the final alignment of wall face, Tatsuoka et al. (1997a) proposed a staged construction procedure, in which a FHR facing is cast-in-place directly on a wrapped-around wall face after most potential deformation of the backfill and the supporting ground has taken place. FHR facing behaves as a continuous beam that is laterally supported by rein-

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forcement layers at many levels with a small vertical spacing of about 30 cm between vertically adjacent reinforcements layers. Consequently, the shear force and moment working at the base of and inside the FHR facing increases basically only in proportion to the wall height H . Therefore, these force values become substantially smaller than those with conventional type RW structures, in particular when the wall height exceeds, say, 10 m. For this reason, the FHR facing could be relatively thin (typically 30 cm) and reinforced only with a small amount of steel reinforcement (Tatsuoka et al., 1997a).

GRS-RWs having a FHR facing performed very well during the 1995 Hyogoken-Nambu Earthquake (e.g., Tatsuoka et al., 1996, 1997b). The performance was substantially better than that of gravity type RWs, while it was equivalent to, or even better than, that of modern cantilever RC RWs, in particular, a GRS-RW with a FHR facing located at Tanata. Tanata was located inside one of the most seriously damaged areas during the earthquake, yet it survived a very high level of seismic load. Similar high performance of GRS RWs during the 1994 Northridge Earthquake was reported (e.g., White and Holtz, 1996). It has been pointed out that the observed high seismic performance of reinforced backfill RWs, including GRS RWs having a FHR facing, have been due to their high dynamic flexibility and high ductility (e.g., Bathurst and Alfaro, 1997; Koseki et al., 1997; Tatsuoka et al., 1998). Despite the above, it is also true that reinforced soil RWs may exhibit relatively large residual shear deformation when subjected to high level seismic loading. For example, the GRS-RW having a FHR facing at Tanata exhibited some noticeable residual shear deformation, about 5%, with a slight outward displacement at the wall base during the earthquake (Tatsuoka et al., 1997b, 1998). A number of other types of reinforced soil structures, including several steel-reinforced soil RWs that survived very strong shaking during the 1995 Hyogoken-Nambu Earthquake, exhibited much larger residual deformation than the GRS RWs having a FHR facing at Tanata and other places (Tatsuoka et al., 1997b).

Conversely, most of the existing modern bridge abutments and piers are RC or steel structures, except for a limited number of abutments of reinforced backfill directly supporting bridge girders. The number of such bridge abutments of GRS RWs having a FHR facing that have been constructed so far in Japan is 18, while the longest bridge girder supported by such a GRS bridge abutment is only 16.5 m, as shown below. Such a limited use is mainly due to the fact that the following points have not been confirmed:

- 1) whether the transient and long-term residual deformation by traffic load of the backfill could be sufficiently small; and
- 2) whether the seismic stability of such reinforced soil structures could be sufficiently high.

Considering that it would be very difficult to fully solve these two potential problems only by increasing the number, length and material rigidity of reinforcement, a new

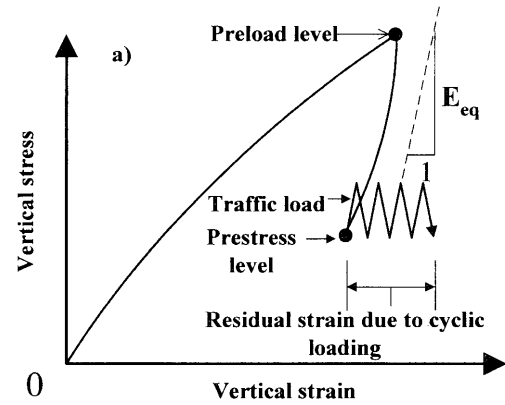


Fig. 1a. Schematic diagram showing the stress-strain behavior of a PLPS GRS structure (not to scale)

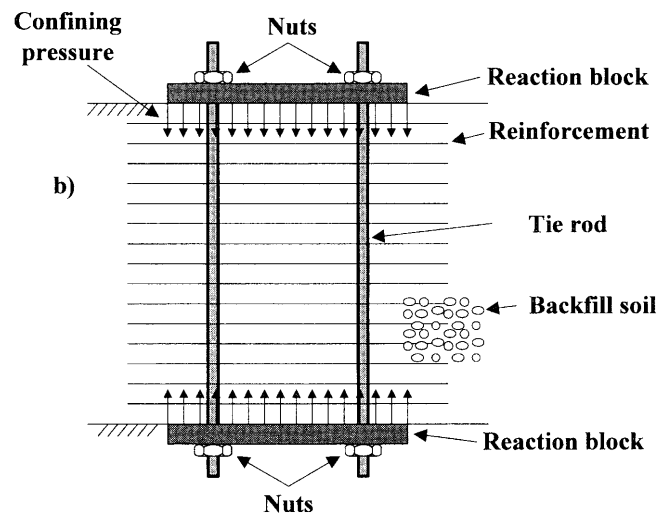


Fig. 1b. Typical cross-section of preloaded and prestressed reinforced soil structure

construction procedure using preloading and prestressing has been proposed (Figs. 1(a) and 1(b); Tatsuoka et al., 1997a and c; Uchimura et al., 1996, 1998 and 2001). In this procedure, a sufficiently large vertical preload is applied to the reinforced backfill by introducing tension into metallic tie rods that penetrate the reinforced backfill with ends fixed to the bottom and top reaction blocks (as shown in Fig. 1(b)). In addition, the stiffness and strength of backfill while the structure is in service is always kept sufficiently high by maintaining sufficiently high prestress. By this preloading and prestressing construction procedure (the PLPS procedure), the transient deformation of reinforced backfill subjected to traffic load can become very small and essentially elastic, and thereby the long-term residual deformation can become very small (i.e., the first issue above). Ketchart et al. (1997) reported the performance of full-scale models of geosynthetic-reinforced soil bridge pier and abutment. In that study, the effects of preloading and prestressing were not evaluated. Adams (1997) proposed applying a large preload to mechanically reinforced backfill, aiming to substantially

The geogrid reinforcement of polyvinyl alcohol coated with polyvinyl chloride (PVC) was used, which has a nominal rupture strength of 73.5 kN/m. This type of grid is widely used to construct GRS retaining walls with a full-height rigid facing. The grid layers were placed with the longitudinal directions of the alternative layers orthogonal to each other and an average vertical spacing between the adjacent layers equal to 15 cm.

- d) A RC block was constructed as the top reaction block on the crest of the completed backfill, which also supported the bridge girders.
- e) Center-hole hydraulic jacks were set between the top reaction RC block and the top ends of the tie rods to vertically preload the backfill. To compress the backfill as much as possible at this stage, maximum preloading is preferred as long as the backfill is not damaged. As the backfill is tensile-reinforced, preload can be applied to a level that is substantially higher than the strength of unreinforced backfill. It is better to maintain the preload as long as possible to allow for as large as possible creep deformation of backfill to occur. The vertical load was then decreased to a prescribed non-zero prestress level such that:
 - (1) the difference between the preload and prestress levels is sufficiently higher than the magnitude of subsequently applied load so that the transient and residual deformation by vertical external load of the backfill could become essentially elastic; and
 - (2) the prestress level is sufficiently high so that the stiffness for essentially elastic deformation of the backfill during when the structure is in service could be sufficiently high.
- f) The top ends of the tie rods were fixed to the top reaction block by using nuts while removing the jacks. After this stage, the prestress remaining in the backfill was in equilibrium with the instantaneous tie rod tension. The tie rod tension can decrease with time as the residual backfill compression increases. It is shown below that the relaxation of the tie rod tension could be significantly reduced by the preloading procedure. This issue is discussed also in Tatsuoka et al. (2000, 2001, 2002a and b). Finally, a lightly steel-reinforced concrete facing (i.e., a FHR facing) was cast-in-place directly on the wrapped-around wall face while firmly connecting the facing to the main body of the reinforced backfill.

The residual settlement of the pier by about 120 train passing per day for a period between the opening of service in the beginning of August 1997 and the end of service (the end of March 2001) was very small, where the train usually consisted of 2 coaches, each weighing about 350 kN without passengers (Fig. 4; Uchimura et al., 2001). A long-term measurement of the tie rod tension and the compression of the pier backfill was started immediately before applying preload on September 5th 1996. The maximum transient compression by each train

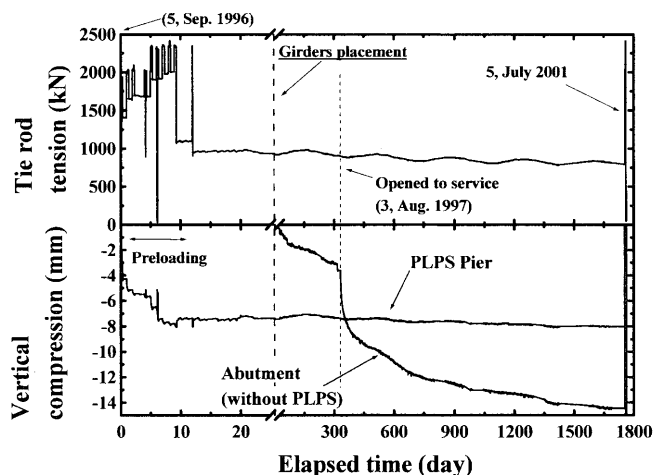


Fig. 4. Time histories of tie rod tension and compression of the GRS pier and compression of the abutment of Maidashi Bridge (Uchimura et al., 2001)

passing of the backfill was only of about 0.025 mm measured by using a pair of sensitive displacement transducers. This small compression means a vertical strain of as small as about 0.001% in the backfill. Very importantly, the deformation of gravel (and sand) is essentially elastic when the strain is smaller than this order of strain amplitude (e.g., Tatsuoka et al., 1999a, b and c). Therefore, one of the keys for high performance of PLPS reinforced soil structure is to keep the transient strain in the backfill as small as above by attaining a high stiffness of backfill.

Abutment A2 (Fig. 2(a)) was constructed in a similar way as the pier by using the same backfill material and reinforcement, but without using the PLPS procedure. A long-term measurement of the compression of the abutment backfill was started immediately after the girder was placed on it. The bridge abutment exhibited a total residual settlement of 12 mm with a maximum transient compression of backfill of about 0.2 mm, which is larger than that of the pier by a factor of about eight, although the maximum train load applied to the abutment was about a half of the value applied to the pier.

To have a better insight into the function of the PLPS procedure and to develop relevant design procedures, a series of laboratory model tests was performed as shown below.

LABORATORY MODEL TEST PROCEDURES

PLPS GRS Bridge Pier Models

A number of rectangular prismatic models of GRS pier were constructed for loading tests, which were 55 cm-high and 35 cm by 35 cm in cross-section (Figs. 5(a) and 5(b)). Each model consisted of 12 layers of about 4.6 cm-thick sub-layers. The backfill soil was air-dried, made of either dense Toyoura sand or a well-graded gravel of crushed sandstone (Fig. 6). The physical properties of those materials are listed in Table 1. The backfill was reinforced with horizontally placed grid layers with a vertical

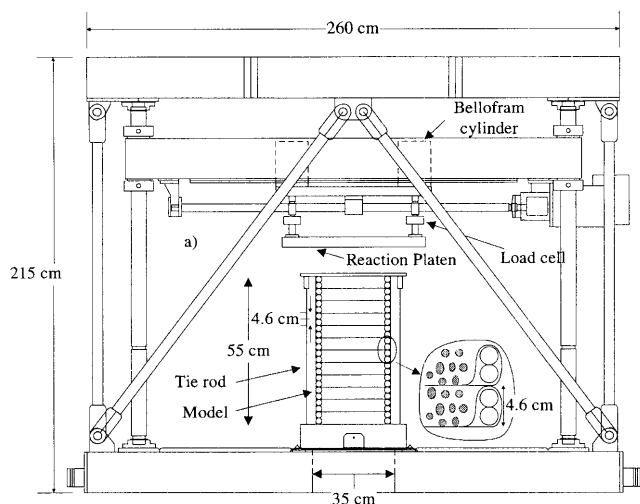


Fig. 5a. General view of the loading system with a model (with four tie rods) set for a loading test

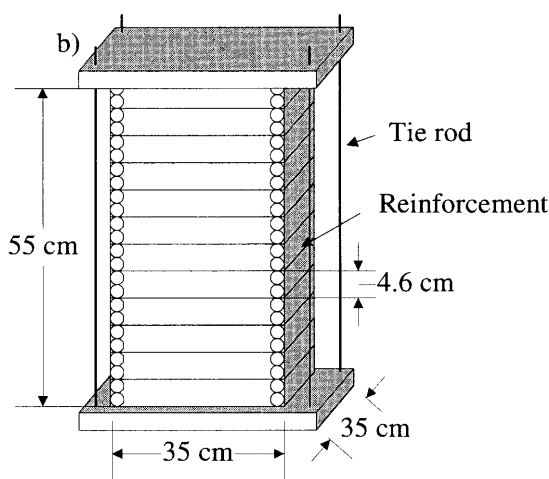


Fig. 5b. Schematic diagram of the test model with four tie rods

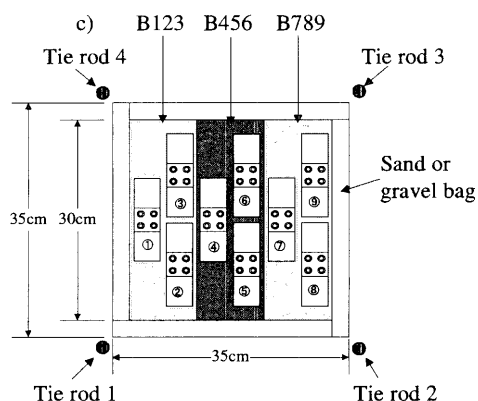


Fig. 5c. Arrangements of load cells in the pedestal at the model

spacing of 4.6 cm. For Toyoura sand, a polymer grid named KJV3000 was used, which was made of polyvinyl alcohol (Vinylon). The thickness, width and length of the grid members are 1 mm, 2 mm and 350 mm, respectively,

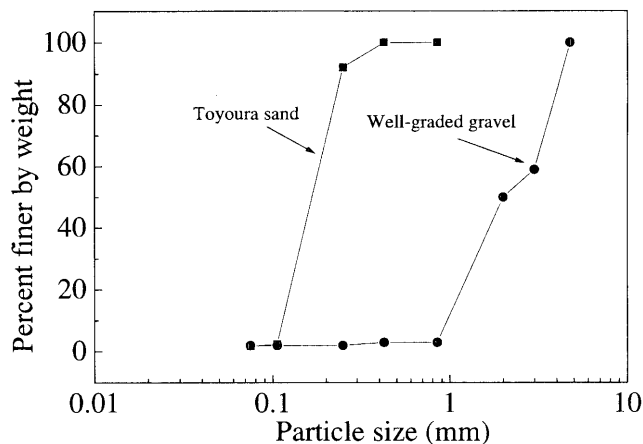


Fig. 6. Grading curves of Toyoura sand and well-graded gravel

Table 1. Physical properties of Toyoura sand and well-graded gravel

	Toyoura sand	Well-graded gravel
U_c	1.44	5.41
D_{50} (mm)	0.2	2.52
D_r (%)	90	90
e_{max}	0.977	0.727
e_{min}	0.597	0.363
ρ_d (g/cm ³)	1.63	1.789

and the aperture is 20 mm. The rupture strength, obtained from tensile tests on single strip specimens at an axial strain rate of 100%/min, is equal to 790 N and a secant stiffness modulus at 5% strain is equal to 10 kN. The strength and stiffness of this grid is approximately half of the one used to construct the bridge abutment and pier described above and many GRS-RWs with a FHR facing (Tatsuoka et al., 1997a). For the gravel, another model grid was used, consisting of 34 phosphor bronze strips (3.5 mm-wide, 0.2 mm-thick and 350 mm-long) for each layer having an average aperture of 7 mm was used. The stiffness of single strip specimen is the order of 82 kN, or 2788 kN per width of the model used in the present study, which is substantially larger than that of the polymer grid.

The periphery of each sub-layer of the backfill was formed by small bags stacked in two layers that were filled with the respective backfill material. The bags were about 4.6 cm in height, and wrapped with the reinforcement grid of Vinylon (see a figure inset in Fig. 5(a)). A steel platen 45 cm by 45 cm in cross-section of 5 cm-thick and with a weight of 282 N was placed on the top of the completed reinforced backfill (Fig. 5(b)). The top ends of the tie rods, when used, were fixed to this platen.

Loading and Measuring Methods

Vertical preload was applied onto the steel platen by using four double-action Bellofram[®] air cylinders

(Fig. 5(a)). After decreasing the vertical load from the preload level to each prescribed prestress level, the prestress during the subsequent cyclic loading stage was either; a) kept constant by using the air cylinders; or b) applied by using four steel tie rods fixed between the top reaction steel platen and the model pedestal (only for some model tests using Toyoura sand) (Fig. 5(b)). In the latter case, to obtain a prescribed initial prestress in the backfill, the vertical load that was first applied to the backfill by using the air cylinders was gradually decreased being replaced with tie rod tension that was applied manually. Each tie rod was equipped with a load cell to measure the tie rod tension. The tie rod tension (i.e., the prestress level) changed when the backfill tended to deform vertically during vertical cyclic loading, while the average value gradually decreased as residual vertical compression of the backfill due to vertical cyclic loading occurred. The tie rod tensions also decreased when the backfill exhibited creep compression. The model tests using tie rods will be termed “tests with tie rods” in subsequent sections.

A total of nine two-component (shear and vertical) load cells were arranged in the pedestal supporting the backfill to measure the stress distribution for an area of 30 cm by 30 cm, inside the sand or gravel bags, at the bottom of the backfill (Fig. 5(c)). The nine load cells were grouped into three load cells in each area (i.e., “B123” and “B789” in the side areas and “B456” in the central area). The settlement at the top reaction platen was measured to obtain average axial (or vertical) strains of the backfill.

Tables 2a and 2b show the test programs. Each test was named, for example, as PL250PS100, which means that the level of preload (PL) was 250 kPa and the level of prestress (PS) was 100 kPa, both defined at the top of the backfill. In each test, 100 cycles of vertical cyclic stresses with double amplitude of 50 kPa were applied with a period of 50 seconds per cycle. The backfill was preloaded and prestressed as follows before cyclic loading:

Toyourea sand (with polymer grid reinforcement):

- 1) Tests to evaluate the effects of preload: Different preload levels, 5 kPa, 100 kPa, 150 kPa, 200 kPa and 250 kPa, were applied for the same prestress equal to 5 kPa. The prestress was applied by the weight of the top reaction platen supplemented by additional load to attain the desired intensity. In test PL5PS5, a prestress level equal to 5 kPa was applied without preloading.
- 2) Tests to evaluate the effects of prestress: Different prestress levels, 5 kPa, 100 kPa and 200 kPa, were applied for the same preload level equal to 250 kPa. In the other two tests, the same preload and prestress levels, equal to 100 kPa or 200 kPa, were used.
- 3) Tests to evaluate the effects of using tie rods: For different combinations of preload and prestress levels, four tie rods were used to confine the backfill during cyclic loading.
- 4) In one test using tie rods, pre-cyclic loading was applied at the preloaded state.

Table 2a. Test program for Toyoura sand backfill (with polymer grid reinforcement)

Name	Preload (kPa)	Prestress (kPa)	Tie rods	Pre-cyclic loading
PL5PS5	5	5	None	None
PL100PS5	100	5	None	None
PL150PS5	150	5	None	None
PL200PS5	200	5	None	None
PL250PS5*	250	5	None	None
PL250PS5*	250	5	None	None
PL250PS100	250	100	None	None
PL250PS200	250	200	None	None
PL100PS100	100	100	None	None
PL200PS200	200	200	None	None
PL100PS100 with tie rods	100	100	Used	None
PL250PS50 with tie rods	250	50	Used	None
PL250PS100 with tie rods	250	100	Used	None
PL250PS150 with tie rods	250	150	Used	None
PL250PS200 with tie rods	250	200	Used	None
PL250PS100 with tie rods and pre-cyclic loading	250	100	Used	Applied

Note: * the same test.

Table 2b. Test program for gravel backfill (with phosphor bronze grid reinforcement; tie rods were not used and pre-cyclic loading was not applied)

Name	Preload (kPa)	Prestress (kPa)
PL5PS5	5	5
PL250PS5	250	5
PL250PS100	250	100
PL250PS200	250	200

Gravel (with phosphor bronze grid reinforcement without using tie rods):

- 1) Two tests to evaluate the effects of preload (PL5PS5 and PL250PS5).
- 2) Three tests to evaluate the effects of prestress (PL250PS5, PL250PS100 and PL250PS200).

The results from the first and second series tests using PL = 5 kPa and 250 kPa are described below in detail, while the results from the other tests are presented when analyzing the data in a summarized form.

TEST RESULTS AND DISCUSSIONS

Effect of Preloading

Figure 7(a) compares the relationships between the average vertical stress acting on the top of backfill and the average vertical strain of backfill from the start of initial

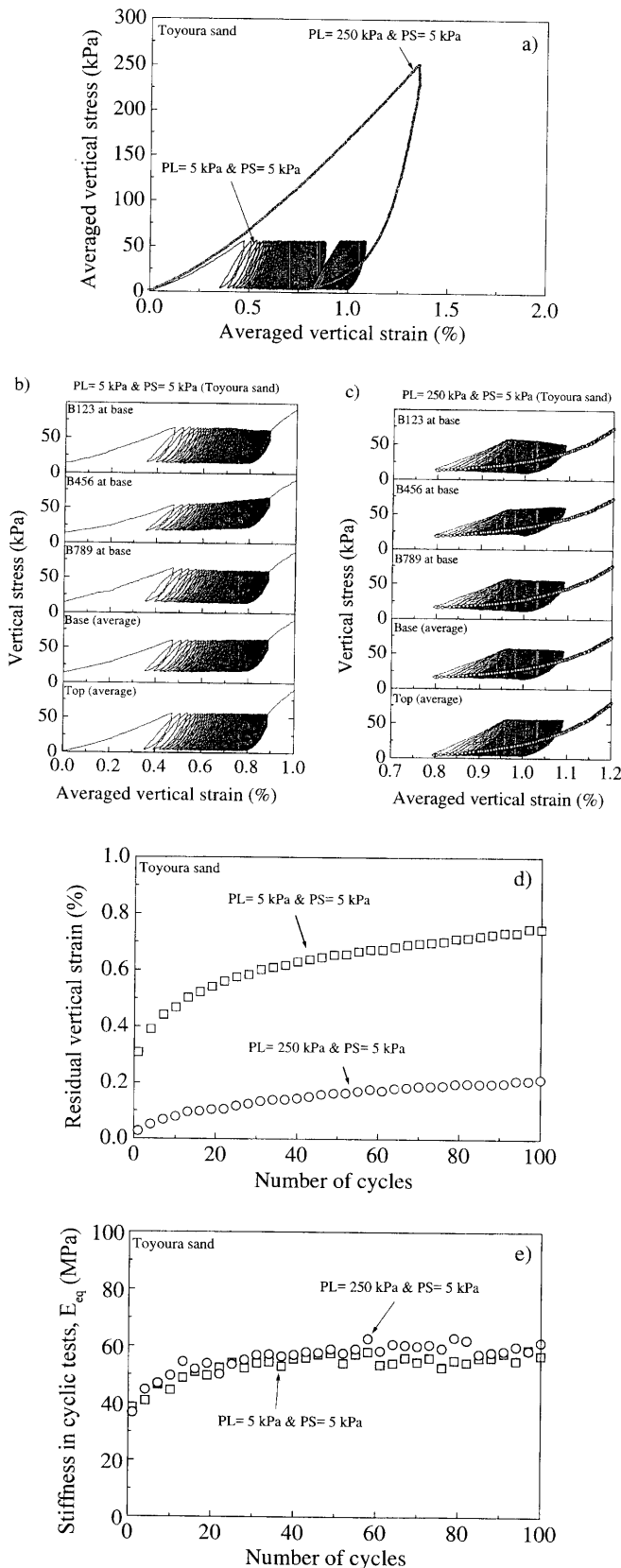


Fig. 7. Effects of preloading on the deformation of backfill during cyclic loading (Toyourea sand without and with preload, not using tie rods): a) overall stress-strain relations, b) and c) relationships between local vertical stress and vertical strain of backfill, b) PL = 5 kPa and PS = 5 kPa, and c) PL = 250 kPa and PS = 5 kPa, d) residual vertical strain and e) stiffness of the backfill during cyclic loading

loading until the end of cyclic loading obtained from two tests, PL250PS5 and PL5PS5, on models of Toyoura sand without tie rods. Figures 7(b) and 7(c) show the local vertical stress in the three areas at the bottom of backfill (B123, B456 and B789), the average vertical stress at the bottom of backfill (inside the sand bags) and the average vertical stress at the top of backfill, plotted against the averaged vertical strain of backfill. Detailed behaviour of each model can be seen from Figs. 7(b) and 7(c). Figures 7(d) and 7(e) compare the residual vertical strains induced by cyclic loading and the equivalent Young's moduli E_{eq} of backfill (both are defined in Fig. 1(a)), plotted against the number of loading cycles, from the two tests. The following trends of behaviour can be seen from these figures:

- 1) The residual compression of backfill by cyclic loading decreased substantially due to the preloading procedure (Fig. 7(d)). This large effect can be attributed to the occurrence of large irreversible strains during the preloading process.
- 2) The backfill rebounded noticeably during unloading from the preload level towards a low prestress level equal to 5 kPa, particularly as the stress level approached this low prestress level (Fig. 7(c)). It is argued later in this paper that this rebound behaviour largely cancelled the positive effects of preloading, enhancing the development of large residual strains of backfill by cyclic loading.
- 3) The E_{eq} value was similar in the two tests (Fig. 7(e)), which means that the effects of preloading on the stiffness of backfill during cyclic loading were negligible.
- 4) The difference in the average vertical stresses between the bottom and top of the backfill was due to the extra load supported by the sand bags and the weight of backfill. As seen from Figs. 7(b) and 7(c), the difference was rather constant during cyclic loading, showing that the vertical load supported by the sand bags did not change noticeably during cyclic loading. However, the local vertical stresses in the side areas (B123 and B789) at the bottom of backfill decreased during cyclic loading although it was at a low rate, while the value in the central area (B456) increased. This result suggests that the backfill located closer to the edge of the model was less stable, by being less constrained by the reinforcement, and therefore it yielded slightly during cyclic loading.

Figures 8(a) through 8(e) show results from two similar tests on the well-graded gravel. The behaviour of the gravel backfill is generally similar to that of the Toyoura sand backfill described above, except for the following. In test PL5PS5 on the gravel (without the preloading history), the average and local vertical stresses at the bottom of backfill (inside the gravel bags) increased noticeably with cyclic loading (Fig. 8(b)). This is due to gradual yielding of the gravel bags during cyclic loading, showing that the gravel bags were less stable than the backfill during cyclic loading, but in a different way from the case with the Toyoura sand backfill.

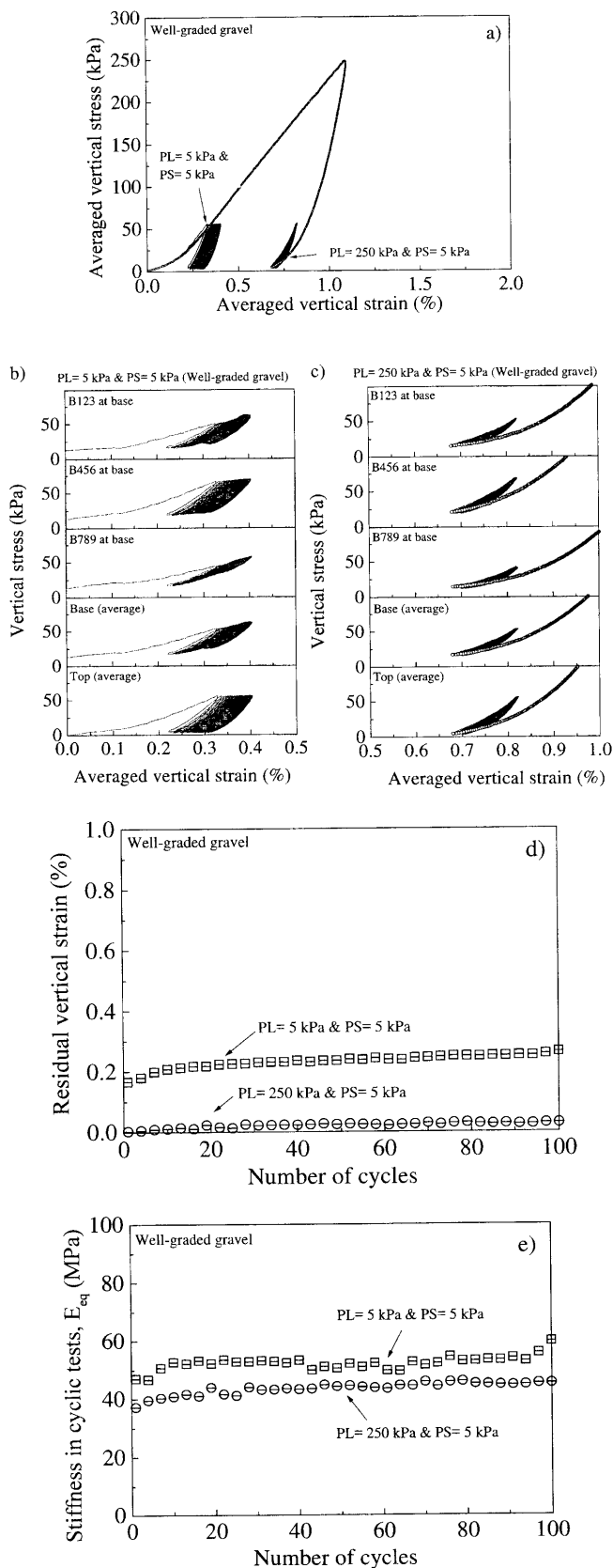


Fig. 8. Effects of preloading on the deformation of backfill during cyclic loading (well-graded gravel without and with preload, not using tie rods): a) overall stress-strain relations, b) and c) relationships between local vertical stress and vertical strain of backfill, b) PL = 5 kPa and PS = 5 kPa, and c) PL = 250 kPa and PS = 5 kPa, d) residual vertical strain and e) stiffness of the backfill during cyclic loading

Effect of Prestress Level

Figure 9(a) compares the overall stress-strain relations of the Toyoura sand backfill obtained from three tests in which the same preload (PL = 250 kPa) was applied while cyclic loading tests were performed at three different prestress levels (PS = 5, 100 and 200 kPa). Figure 7(b) and Figs. 9(b) and 9(c) show the detailed behaviour of backfill during cyclic loading in the three tests, while Figs. 9(d) and 9(e) show the residual strain and the equivalent Young's modulus E_{eq} , plotted against the number of loading cycles. The following trends of behaviour can be seen from these figures:

- 1) Among the three tests, the smallest residual compression of backfill by cyclic loading was observed in test PL250PS100, in which the vertical stress applied to the backfill was reduced from 250 kPa to 100 kPa before cyclic loading.
- 2) The residual compression of backfill by cyclic loading was similar in the other two tests, PL250PS5 and PL250PS200. However, the compression of backfill in these two tests was much larger than test PL250PS100. This trend of behaviour could be explained as follows:
 - a) In test PL250PS5, it seems that relatively large irreversible strains were developed due to the following two factors. First, the strain amplitude in each loading cycle was relatively large due to small stiffness of backfill because of a relatively low vertical stress (Fig. 9(e)). In this respect, the following equation is known to be relevant for the elastic Young's modulus E_v , defined for vertical elastic strain increments (usually lower than about 0.001% for unbound granular materials; e.g., Tatsuoka et al., 1999a):

$$E_v = C \cdot f(e) \cdot \left(\frac{\sigma_v}{\sigma_0} \right)^m \quad (1)$$

where C is the material parameter; $f(e)$ is the void ratio function; σ_0 is the reference pressure; and m is the power, equal to around 0.5. As the vertical strain amplitude in these tests was relatively small, ranging from about 0.01% to about 0.05%, the E_{eq} values should have the similar feature as the elastic Young's modulus represented by Eq. (1). As the amplitude of cyclic stresses was always the same in the present test, a smaller elastic Young's modulus resulted in a smaller equivalent modulus during cyclic loading. This trend of behaviour was enhanced by the effects of strain-non-linearity of stiffness associated with large strains due to smaller stiffness values. Second, relatively large vertical swelling took place in the backfill when the vertical stress in the backfill approached this low prestress level during the unloading process, which may have destabilized the sand micro-structure that had developed during the preloading process. This point is analysed in detail later in this paper.

- b) In test PL250PS200, the prestress was higher than

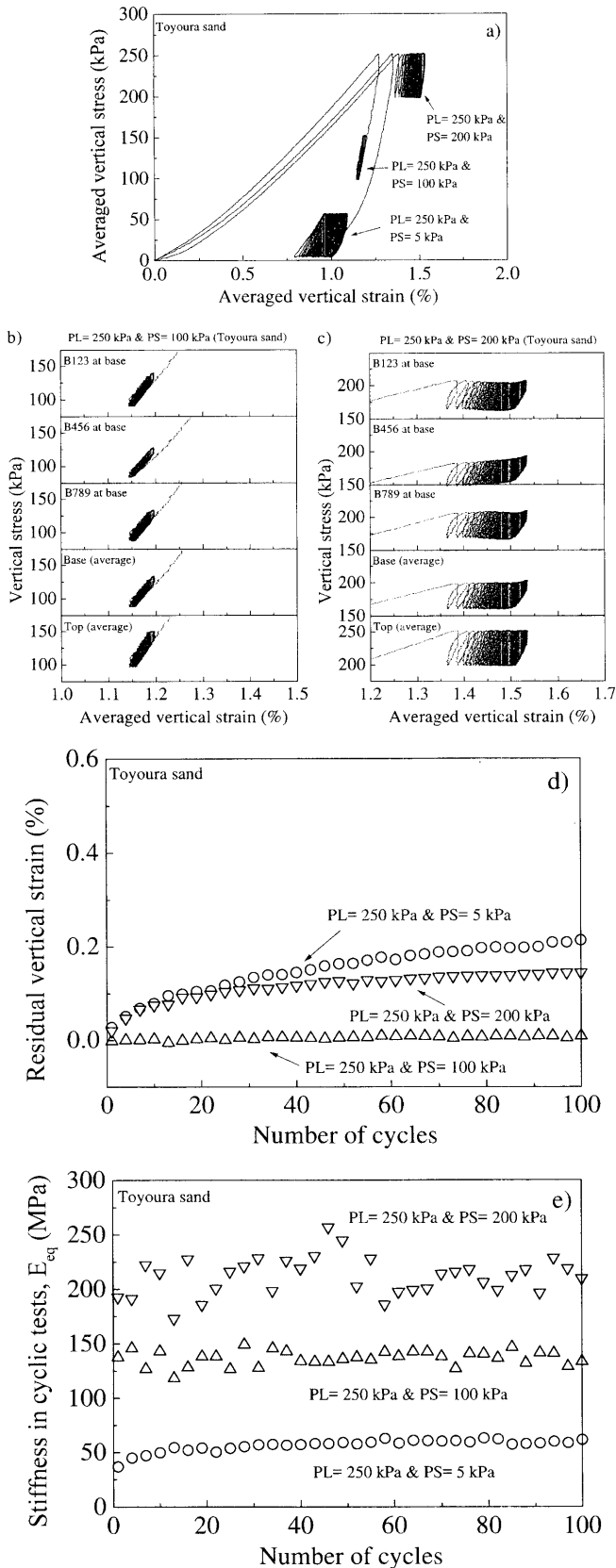


Fig. 9. Effects of prestressing on the deformation of backfill during cyclic loading (Toyoura sand with the same preload, not using tie rods): a) overall stress-strain relations, b) and c) relationships between local vertical stress and vertical strain of backfill, b) PL = 250 kPa and PS = 100 kPa, and c) PL = 250 kPa and PS = 200 kPa, d) residual vertical strain and e) stiffness of the backfill during cyclic loading

in the other two tests, resulting in the largest E_{eq} value (Fig. 9(e)). The benefit by this factor was totally masked by the development of relatively large irreversible strains by yielding of the backfill during cyclic loading, which occurred because the peak vertical stress during cyclic loading was the same as the preload level, equal to 250 kPa.

- 3) In test PL250PS200, in the three areas at the bottom of backfill, the local vertical stress increased slightly with cyclic loading (Fig. 9(c)). This behaviour would be due to gradual yielding of the sand bags during cyclic loading, as observed in test PL250PS5 on the gravel backfill (Fig. 8(b)).

Figures 10(a) through 10(e) show results obtained from three similar tests on the gravel backfill. Basically the same trends as those seen in the tests with Toyoura sand backfill, described above, can be seen from these figures. However, a relatively large scatter is seen in the primary loading stress-strain relations of the gravel backfill (Fig. 10(a)), compared to those of the Toyoura sand backfill (Fig. 9(a)). This would be due partly to a possibly relatively large heterogeneity in the density and stress-strain property within each gravel backfill model because of larger particle sizes of gravel as compared to the model dimensions and the aperture of the reinforcement. The other variance observed in the tests on the gravel is that the E_{eq} value in test PL250PS200 was slightly smaller than that in test PL250PS100 despite a higher prestress in the former test. This could be due to a much larger amount of yielding of backfill, resulting in a large destabilisation of the micro-structure of gravel, during cyclic loading in test PL250PS200 than in test PL250PS100.

In Figs. 11(a) and 12, the residual vertical strain at a cycle of 100 from all the tests in which tie rods were not used is plotted against the respective normalized amount of unloading from the preload level σ_{PL} to the prestress level σ_{PS} , defined as:

$$UR = \frac{\sigma_{PL} - \sigma_{PS}}{\sigma_{PL}} \quad (2)$$

The ultimate residual vertical strain that would be attained at the infinite number of loading cycles (N_c) in each test was estimated by fitting a hyperbolic relationship to the respective relationship between ϵ_r and N_c . This value is also plotted in Figs. 11(a) and 12. It can be reconfirmed from these figures that for both Toyoura sand and the well-graded gravel, the smallest residual strain by cyclic loading was obtained when UR was around 0.5. Note that this behaviour is consistent with the actual performance of the prototype reinforced soil structure (Fig. 2). The plastic swelling strain (ϵ^p)_{swelling} shown in Figs. 11(a) and 12 is explained later in this paper.

The effects of different combination of PL and PS were not noticeable on the relationships presented in Fig. 11, mainly because of a relatively small range of test conditions and small number of tests performed. However, the test results presented in Fig. 11(b) suggest that for the same value of UR , the residual strain decreases with an

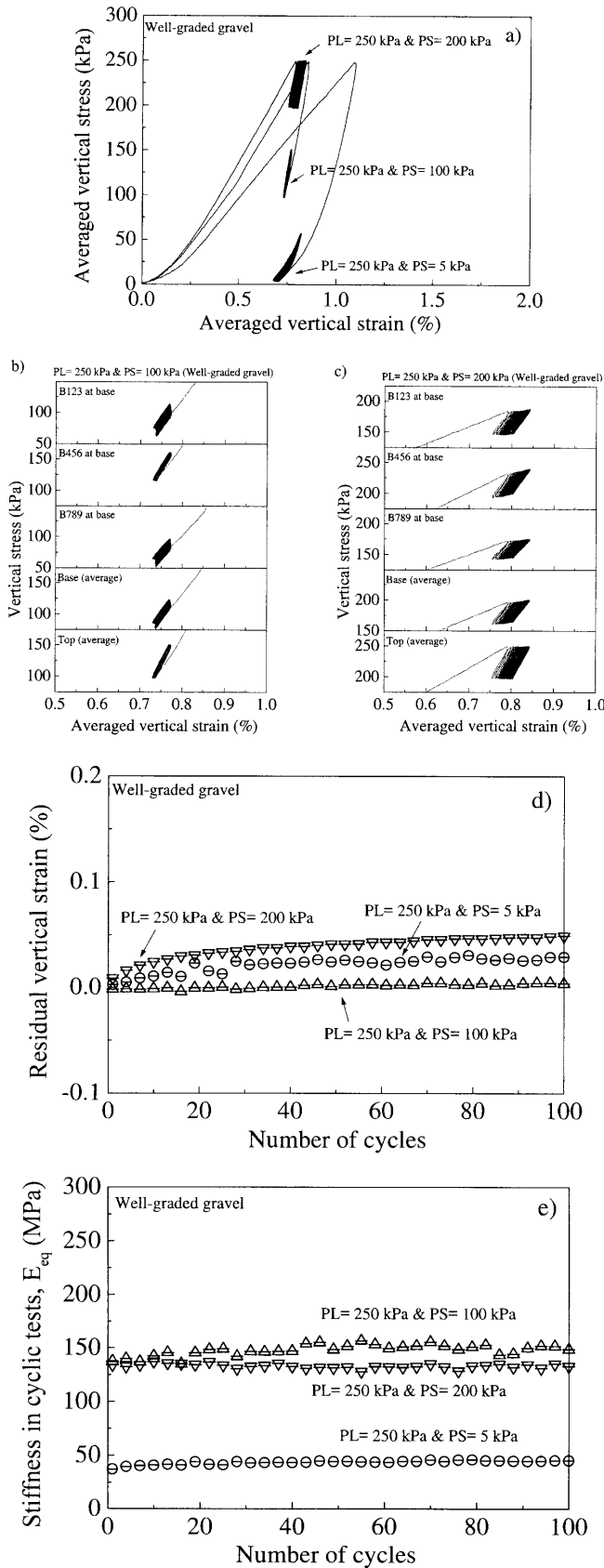


Fig. 10. Effects of prestressing on the deformation of backfill during cyclic loading (well-graded gravel with the same preload, not using tie rods): a) overall stress-strain relations, b) and c) relationships between local vertical stress and vertical strain of backfill, b) PS = 100 kPa and 200 kPa, and c) PS = 100 kPa and 200 kPa, d) residual vertical strain and e) stiffness of the backfill during cyclic loading

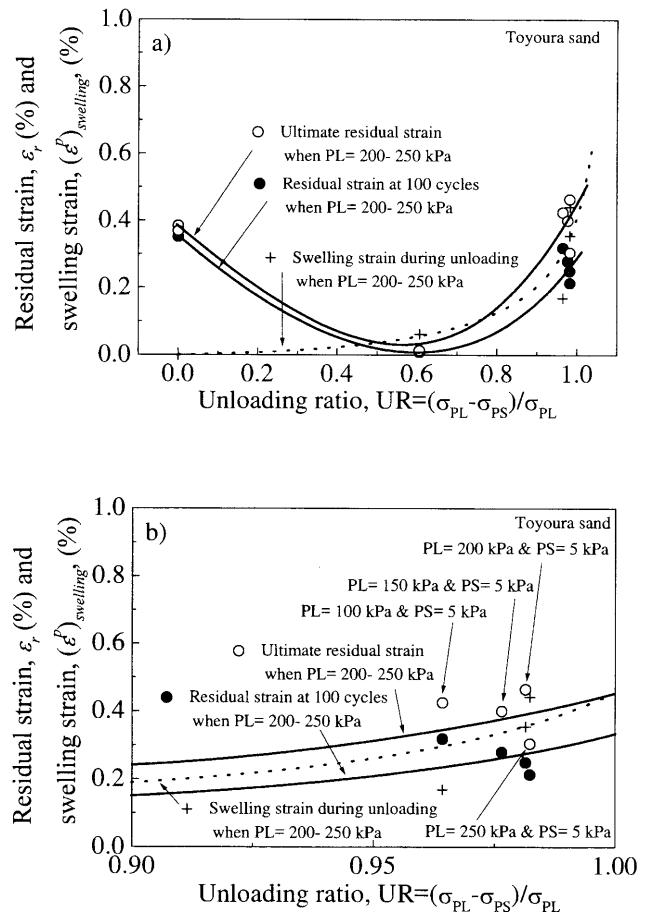


Fig. 11. Summary of residual strains at a cycle of 100 and ultimate state together with the swelling axial strain during unloading (Toyoura sand): a) for a full range of UR and b) for UR = larger than 0.9

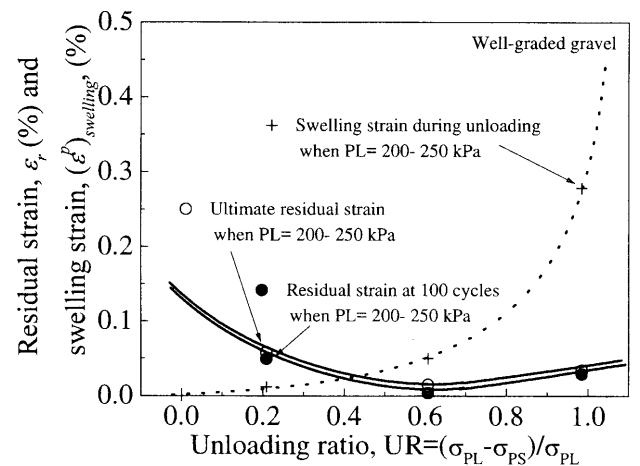


Fig. 12. Summary of residual strains at a cycle of 100 and ultimate state together with the swelling axial strain during unloading (gravel)

increase in the preload level.

In summary, to achieve a reasonably high stiffness as well as very small residual compression of reinforced backfill subjected to a very large number of relatively small-amplitude cyclic stresses, it is effective to apply a

sufficient amount of preload to the backfill and to maintain an appropriate amount of prestress while in service. The test results presented above indicate that the residual compression of backfill by cyclic loading could become very small when the prestress level is around a half of the preload level, and the difference between the preload and prestress levels is more than about two times the amplitude of cyclic stresses (i.e., the test conditions of test PL250PS100). Among the different tests reported so far in this paper, the test conditions of test PL250PS100 simulate most closely the PLPS procedure employed for the prototype GRS bridge pier described in Fig. 2, indicating that the PLPS procedures used in the field was appropriate.

Combined Effects of Backfill Soil Type and Reinforcement Type

By comparing Figs. 7 and 8, the following combined effects of backfill soil type and reinforcement type can be seen:

- 1) The compression of the gravel backfill reinforced with a phosphor bronze grid during the primary loading was noticeably smaller than that of the Toyoura sand backfill reinforced with a polymer grid.
- 2) Although the swelling of backfill at the initial stage of the unloading process from the preload level was similar between the Toyoura sand and gravel backfills, the swelling of the gravel backfill when the vertical stress approached a prestress level of 5 kPa was much smaller than that of the Toyoura sand backfill. The residual compression by cyclic loading of the gravel backfill was also much smaller than that of the Toyoura sand backfill in both cases with and without the preloading process.
- 3) In contrast to the trends of behaviour 1) and 2) above, the equivalent Young's modulus E_{eq} of the gravel backfill was slightly smaller than that of the Toyoura sand backfill.

The reasons for this apparent inconsistency are not known. One of the possible causes for the relatively low E_{eq} values of the gravel backfill would be relatively large effects of bedding error at the interface between the gravel and the reinforcement. This is because a less perfect fitting between the reinforcement and the backfill is possible for a coarser backfill material.

There are the following two possible causes for different behaviour between the two types of backfill soil reinforced with different types of reinforcement;

- 1) different reinforcement materials (a polymer geogrid for Toyoura sand versus a phosphor bronze grid for the gravel); and
- 2) differences in the grading characteristics, the shape and crushability of particles, and the dry density between the two soil types.

Another series of similar model tests was performed very recently using the same type of gravel as used in the present study while using a similar polymer grid in the authors' laboratory (Shibata and Hirakawa, 2001). The

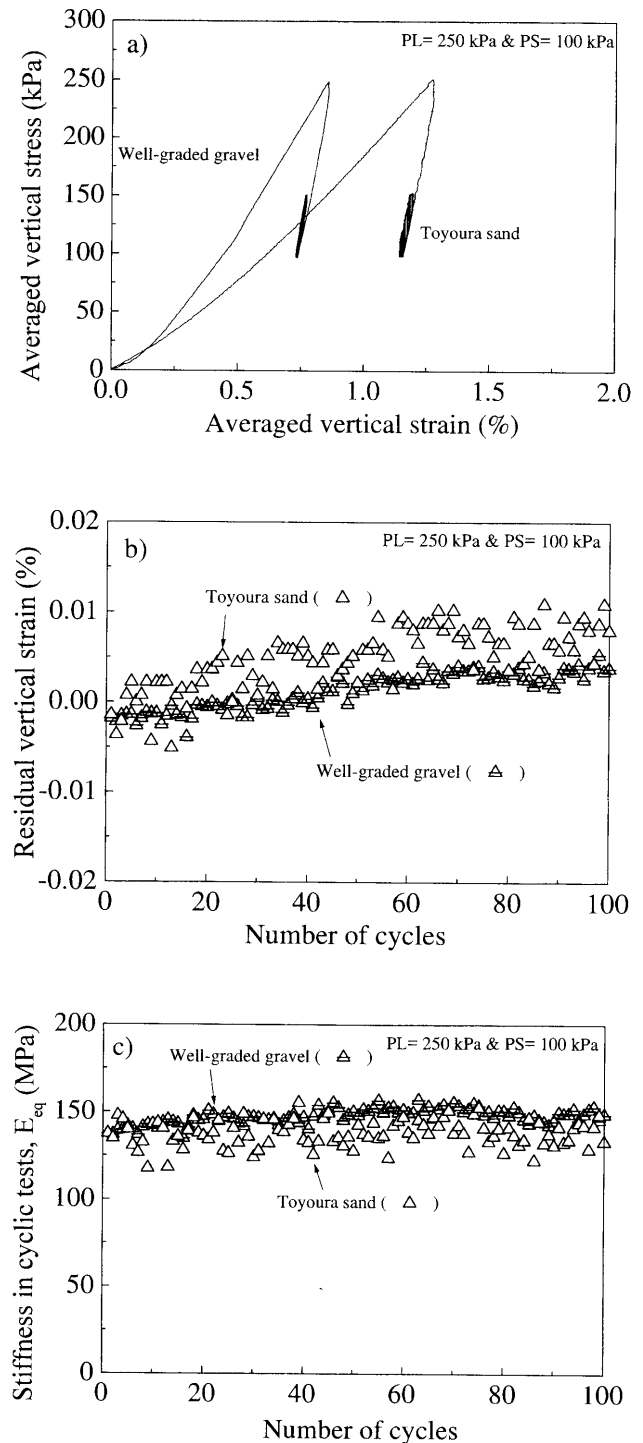


Fig. 13. Effects of backfill soil type: a) overall stress-strain relations, b) residual vertical strain of the backfill and c) stiffness of the backfill during cyclic loading

test results indicated that both of the two factors are relevant to the difference in the behaviour between the backfills of Toyoura sand and the gravel observed in the present study. This point will be reported in the very near future.

Figures 13(a) through 13(c) compare results from cyclic loading tests on the backfill of Toyoura sand reinforced with a polymer grid and the gravel backfill reinforced

with a phosphor bronze grid for the case of PL = 250 kPa and PS = 100 kPa, which is the best combination of preload and prestress levels employed in the present study. It may be seen from Fig. 13(b) that under this test condition, the combined effect of backfill type and reinforcement type on the residual compression of backfill by cyclic loading was small. This test result indicates that the combined effects of soil type and reinforcement type on the behaviour of backfill during cyclic loading could become small when the backfill is highly compacted and an appropriate combination of preload and prestress is used. It should be noted however that prototype PLPS reinforced soil structures will be subjected to a substantially larger number of loading cycles during its life span than the number of loading cycles applied in the present study, equal to only 100. Therefore, a small difference in the residual vertical strain seen in Fig. 13(b) could eventually result in a large difference after a much larger number of loading cycles. To keep the residual vertical compression of reinforced backfill as small as possible, therefore, it can be recommended to use such a well-graded gravel as the one used in the present study, or a better-graded one as the one (with $D_{50} = 0.9$ mm and $U_c = 16.5$; Uchimura et al., 1998) used for the prototype reinforced soil structure presented in Fig. 2. Another important reason for this recommendation is that in another series of model tests in which models were subjected to dynamic load (Shinoda, 2001a), the transient and residual deformation of the reinforced backfill of the gravel used in the present study was substantially smaller than that of the backfill of Toyoura sand.

Effect of Using Tie Rods

With the prototype PLPS GRS structure (Fig. 2), the preload was applied to the backfill and the initial prestress was set by using hydraulic jacks attached to the top ends of four tie rods, while the prestress was maintained by using four tie rods for one year and for about three and a half years before and after the structure was opened to service, respectively. It is very likely that if the backfill had exhibited a noticeable residual compression under this condition, the tie rod tension would have decreased to a much larger extent. To confirm this speculation, two additional tests were performed using tie rods as the prototype GRS structure, as presented below.

Figures 14(a) through 14(d) compare the behaviour during cyclic loading of Toyoura sand backfill in two similar tests with and without using tie rods while employing the same PLPS procedure (PL = 250 kPa and PS = 100 kPa). Test PL250PS100 with tie rods more closely simulates the prototype procedure used for the prototype structure (Fig. 2) than test PL250PS100 without tie rods. In test PL250PS100 with tie rods, a noticeable delayed recovery in the average axial strain took place at a constant vertical stress for a period between the end of unloading and the start of cyclic loading. For this period, the vertical load applied by using air cylinders was replaced with tie rod tension introduced manually. It may be seen from Figs. 14(b) and 14(c) that within the number

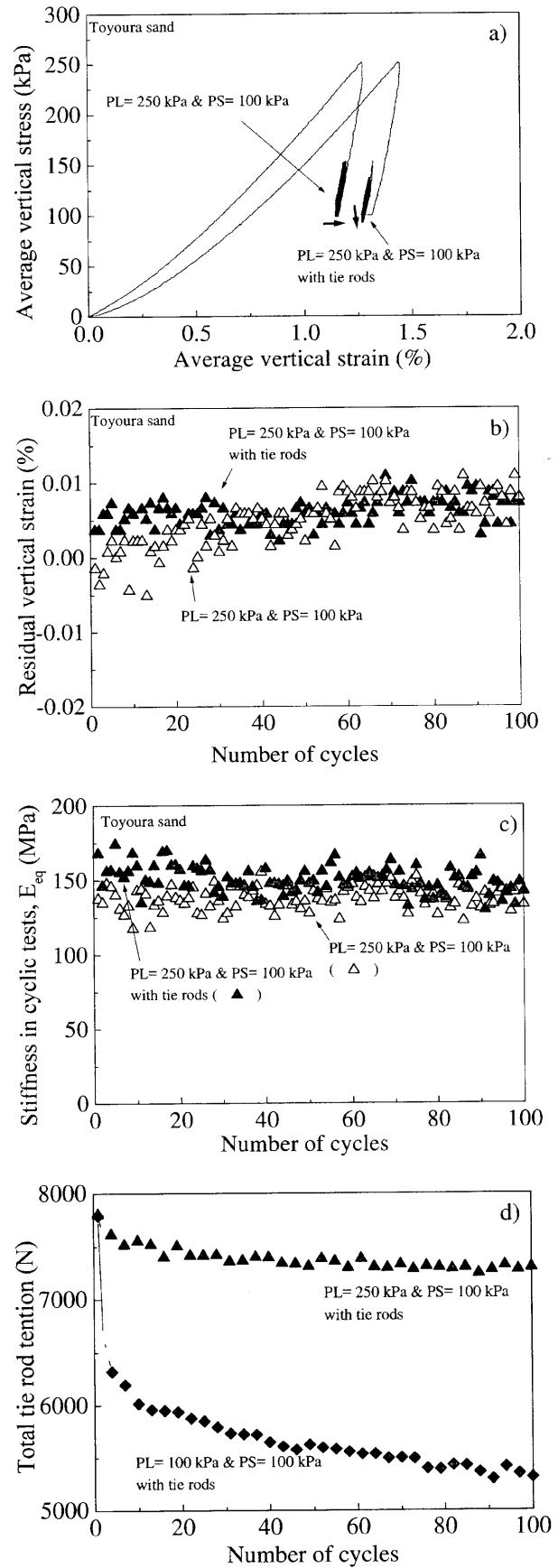


Fig. 14. Effects of using tie rods (PL = 250 kPa and PS = 100 kPa): a) overall stress-strain relations, b) residual vertical strain, c) stiffness of the backfill during cyclic loading and d) average prestress plotted against the number of loading cycles

of loading cycles applied in the present study (i.e., 100 cycles), the transient and residual deformation of backfill in the two tests was similar and acceptably small. This small residual compression of backfill by cyclic loading corresponds to the creep recovery phenomenon observed immediately before the start of cyclic loading described above. In the test with tie rods, however, the tensile stress in the tie rods (i.e., the prestress) decreased gradually with cyclic loading due to residual compression of backfill (Fig. 14(d)). Therefore, some measures to prevent such a reduction in the tie rod tension may become necessary to take when the structure is used for a long time.

It can also be seen from Fig. 14(d) that the decreasing rate of tie rod tension with cyclic loading was much more significant in a similar test without preloading (test PL100PS100 with tie rods). This behaviour was due to relatively large residual compression of backfill by cyclic loading (see Figs. 15(a) and 15(b)). In this test, the compressive strain of backfill increased by creep deformation for a period when the vertical stress that had been applied by using air-cylinders was replaced with tension introduced manually in the tie rods. The residual strain by cyclic loading was much larger in this test than that in the test with preloading, corresponding to the noticeable creep deformation property described above in test PL100PS100 with tie rods and the noticeable creep recovery described before in test PL250PS100 with tie rods. For the same reason as above, the residual settlement by cyclic loading was also very large in test PL100PS100 without tie rods.

It may also be seen from Fig. 15(b) that in the case without preloading, the residual strain of backfill became much smaller by using tie rods. This was because, when the prestress was maintained with tie rods, the vertical stress in the backfill decreased with the increase in the residual compression of backfill, which gradually unloaded (or preloaded) the backfill. However, this benefit will be eventually lost when the prestress ultimately becomes too low after a large number of loading cycles, resulting in a too small stiffness and strength of backfill. This condition is similar to that for a test in which a relatively high PL value was applied, while the initial PS value had been made very low (as test PL250PS5, presented in Fig. 9). Then, the residual compression of backfill by cyclic loading will become too large. In addition, when the prestress has become too low, the seismic stability of structure becomes very low (Shinoda et al., 2001b).

The slope of the top and bottom envelopes of the relationships between the average vertical stress and the vertical strain of the backfill in the tests using tie rods seen in Figs. 14(a) and 15(a) is equal to the stiffness of the tie rods (Tatsuoka et al., 1997c). Therefore, by using softer tie rods, the decreasing rate of tie rod tension associated with compression of backfill becomes smaller. In the paper by Shinoda et al. (2001b), it is shown that the use of a relatively soft tie rod system (together with a ratchet mechanism to prevent the expansion or rebound of backfill) can help achieve a very high seismic stability of reinforced soil structure.

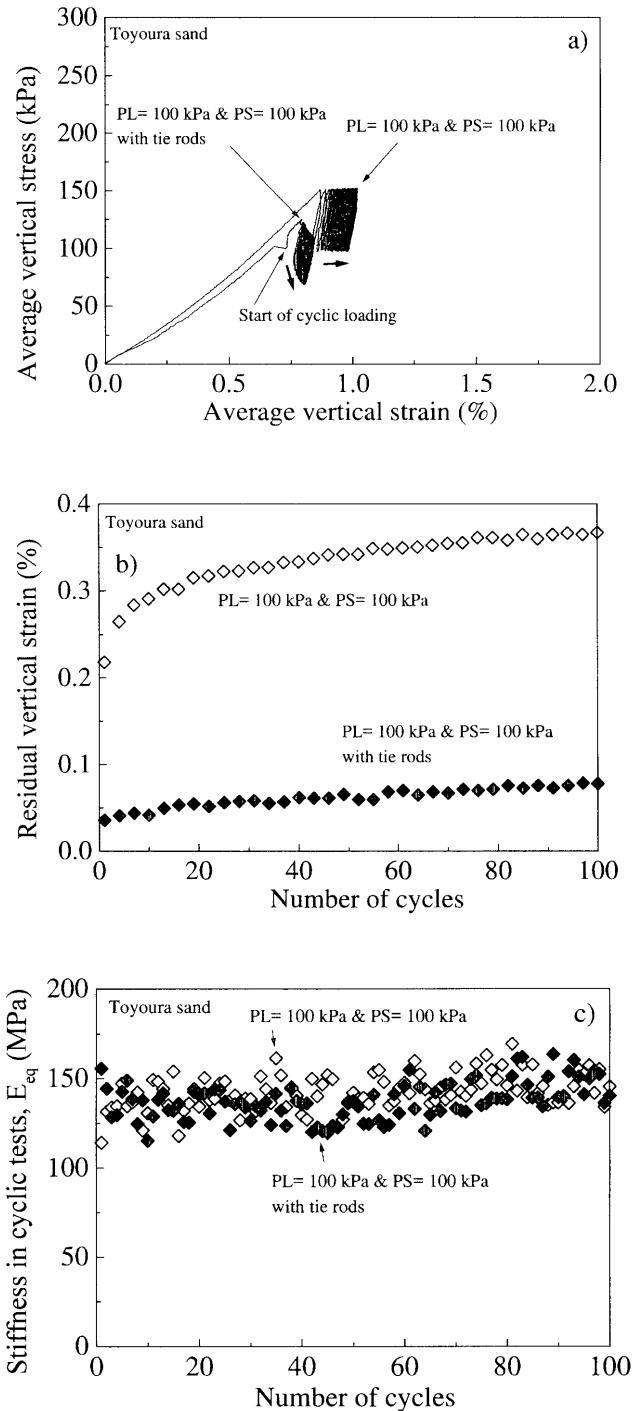


Fig. 15. Effects of using tie rods (PL = 100 kPa and PS = 100 kPa): a) overall stress-strain relations, b) residual vertical strain and c) stiffness of the backfill during cyclic loading

Effect of Pre-Cyclic Loading at Preloaded State

In the last test with Toyoura sand, effects of pre-cyclic loading at the preloading stage were evaluated. As shown in Fig. 16(a), 100 cycles of cyclic vertical stresses with a double amplitude of 50 kPa were applied at the preloaded state before unloading to prestress level of 100 kPa (i.e., test PL250PS100 with tie rods and pre-cyclic loading). In this test, the tie rods were set before applying cyclic loads

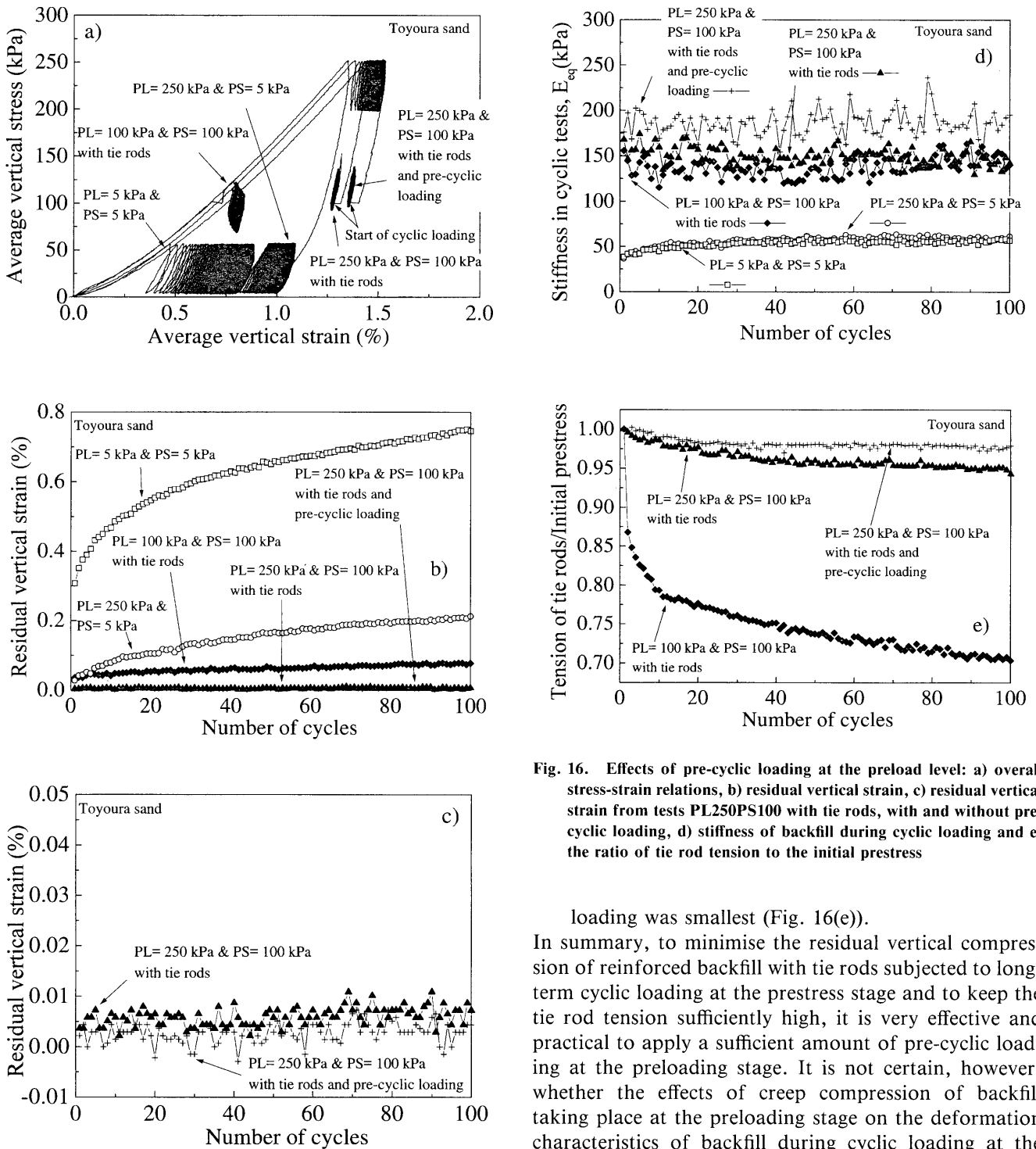


Fig. 16. Effects of pre-cyclic loading at the preload level: a) overall stress-strain relations, b) residual vertical strain, c) residual vertical strain from tests PL250PS100 with tie rods, with and without pre-cyclic loading, d) stiffness of backfill during cyclic loading and e) the ratio of tie rod tension to the initial prestress

at the prestress level, in the same way as the other tests with tie rods. Figures 16(a) through 16(e) compare the behaviour of backfill from this test with those from several other representative tests with Toyoura sand. The following trends of behaviour in this last test may be seen from these figures:

- 1) The residual strain of backfill by cyclic loading was smallest among all the tests (Figs. 16(b) and 16(c)).
- 2) The stiffness E_{eq} at the cyclic loading stage was largest (Fig. 16(d)).
- 3) The decreasing rate of tie rod tension with cyclic

loading was smallest (Fig. 16(e)).

In summary, to minimise the residual vertical compression of reinforced backfill with tie rods subjected to long-term cyclic loading at the prestress stage and to keep the tie rod tension sufficiently high, it is very effective and practical to apply a sufficient amount of pre-cyclic loading at the preloading stage. It is not certain, however, whether the effects of creep compression of backfill taking place at the preloading stage on the deformation characteristics of backfill during cyclic loading at the prestressed stage are equivalent to those by pre-cyclic loading at the preloading stage. Further study into this issue is needed. On the other hand, Shibata et al. (2001) has recently found that under otherwise the same test conditions as “test PL250PS100 with tie rods and pre-cyclic loading”, the residual strain by cyclic loading becomes substantially smaller to nearly zero, or even a negative value, when creep recovery is not allowed to take place before applying cyclic loads at the prestress state. This was also the actual practice with the PLPS GRS bridge pier described in the preceding section.

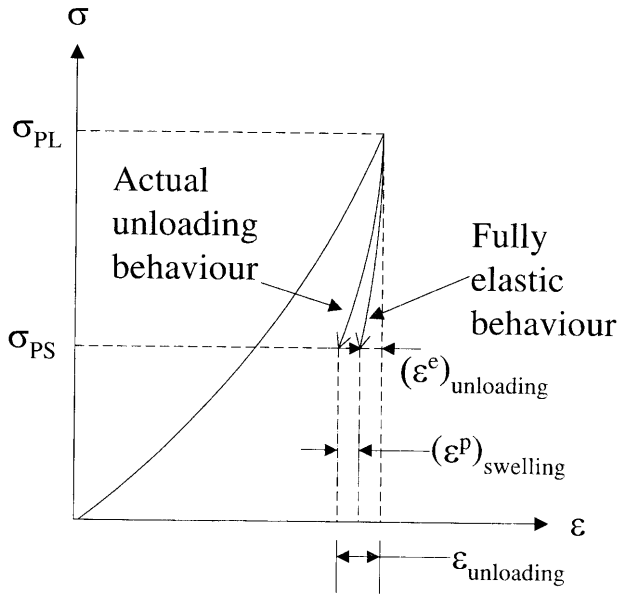


Fig. 17. Definitions of vertical strains

NEGATIVE EFFECTS OF SWELLING DURING UNLOADING

Evaluation of Elastic Strain during Unloading

Considering that irreversible (i.e., inelastic) strains could change the micro-structure of unbound soil, it is likely that irreversible vertical swelling taking place during the unloading process from a preload level to a prestress level will have negative effects on the behaviour of backfill during subsequent cyclic loading performed at the prestress state. This point is analysed below.

Figure 17 defines the following various vertical strains:

- a) $\epsilon_{unloading}$ (positive for swelling) is the total vertical strain that takes place during the unloading process from the preloading state, where $\sigma_{PL} = 250$ kPa in the present case, to the prestress state where $\sigma = \sigma_{PS}$;
- b) $(\epsilon^e)_{unloading}$ (positive for swelling) is the elastic strain that takes place during the unloading process, which is obtained as:

$$(\epsilon^e)_{unloading} = - \int_{\sigma_{PL}}^{\sigma_{PS}} \frac{d\sigma_v}{E_v} = \int_{\sigma_v = \sigma_{PS}}^{\sigma_{PL}} \frac{d\sigma_v}{E_v} \quad (3)$$

where E_v is the elastic Young's stiffness of reinforced backfill, which is a function of the instantaneous vertical stress σ_v (as Eq. (1)); and

- c) $(\epsilon^p)_{swelling}$ (positive for swelling) is the plastic swelling vertical strain that takes place during the unloading process, which is equal to " $\epsilon_{unloading} - (\epsilon^e)_{unloading}$ ".

The values of $(\epsilon^e)_{unloading}$ and $(\epsilon^p)_{swelling}$ were obtained as follows. Figures 18(a) and 19 show the relationships between the E_{eq} value (averaged for cycles 20–100) and the vertical stress σ_v at which the respective E_{eq} value was evaluated, obtained from all the cyclic loading tests on the backfill of Toyoura sand and the gravel performed in the present study (i.e., solid square data points). The following Eq. (4), which is similar to Eq. (1) for E_v , was fitted to the data points:

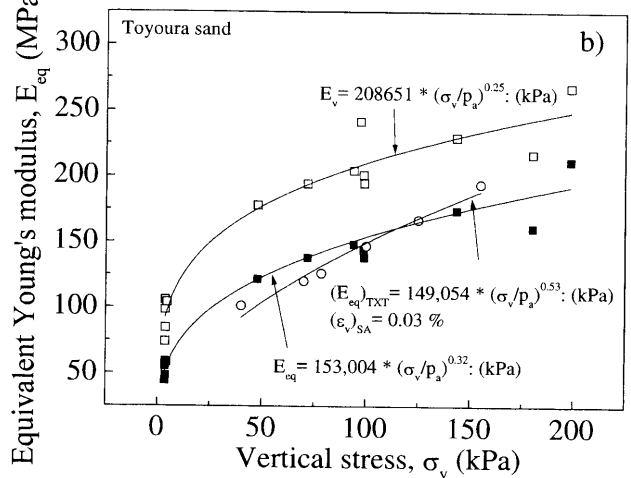
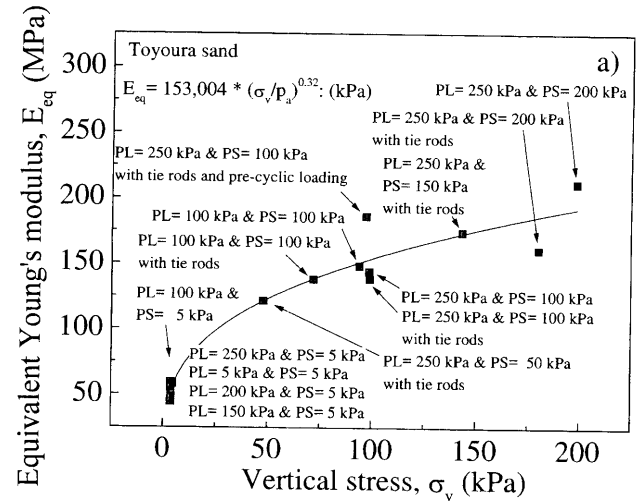


Fig. 18. Stiffness E_{eq} during cyclic loading plotted against the vertical stress: a) E_{eq} from model tests and b) E_{eq} from model tests and cyclic triaxial tests and the elastic stiffness E_v of backfill used in the analysis (Toyouira sand)

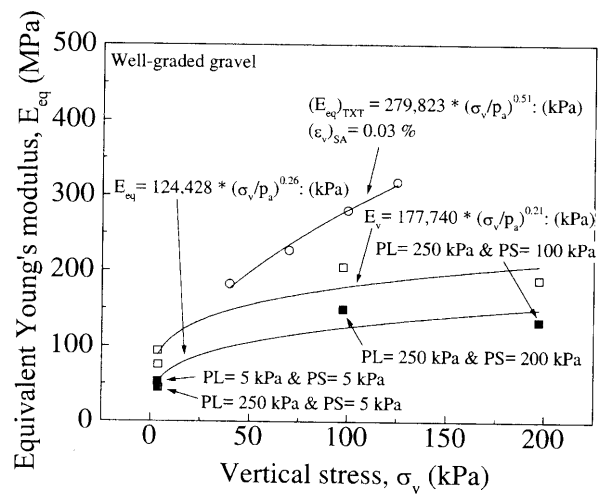


Fig. 19. Stiffness E_{eq} during cyclic loading from model tests and cyclic triaxial tests and the elastic stiffness E_v of backfill used in the analysis, plotted against the vertical stress (well-graded gravel)

$$E_{eq} = A \cdot \left(\frac{\sigma_v}{p_a} \right)^m \quad (4)$$

where p_a is the reference pressure which is equal to 98 kPa. The best fitted values of the parameters are; $A = 153,004$ and $m = 0.32$ for Toyoura sand (Fig. 18(a)); and $A = 124,428$ and $m = 0.26$ for the well-graded gravel (Fig. 19). As the single amplitude of average vertical strain $(\epsilon_v)_{SA}$ in the backfill during cyclic loading ranged from about 0.01% to about 0.05% (on average around 0.03%), these E_{eq} values should be noticeably smaller than the elastic Young's modulus E_v , defined for $(\epsilon_v)_{SA} =$ around 0.001% or less, of the backfill. The E_v values of reinforced backfill to be used in Eq. (3) were obtained as follows.

Figures 20(a) and 20(b) show the relationships between the equivalent Young's modulus E_{eq} and the single amplitude of axial strain $(\epsilon_v)_{SA}$ obtained from cyclic triaxial tests at different values of σ_v while at a constant $\sigma_h (= 40$ kPa) on air-dried specimens of air-pluviated Toyoura sand with an initial void ratio of 0.661 (= a relative density of 89.9%) and gravel with an initial void ratio of 0.561 (= a relative density of 84.2%). The specimens were first isotropically consolidated to $\sigma_v = \sigma_h = 40$ kPa and then cyclic vertical stresses were applied additively to the vertical stress at the initial isotropic or triaxial compression stress state (so the stress state during cyclic loading was always under triaxial compression stress conditions). It may be seen that the E_{eq} values for a strain range of around 0.01% and 0.01% increase with increase in the vertical stress σ_v ; i.e., the E_{eq} value is a function of $(\sigma_v/p_a)^m$, where power m is about 0.58. This behaviour is consistent with Eq. (1), which was derived based on the fact that the elastic Young's modulus E_v defined for vertical strains of less than about 0.001% for clean sands is essentially a unique function of σ_v (e.g., Hoque and Tatsuoka, 1998; Tatsuoka et al., 1999a and b). A similar trend of behaviour can be seen in the E_{eq} values obtained from the model tests (Figs. 18(a) and 19).

From the triaxial test data presented in Figs. 20(a) and 20(b), the values of E_{eq} at the average value of $(\epsilon_v)_{SA}$ in the model tests, equal to 0.03%, were estimated. In so doing, the relationships between E_{eq} and $(\epsilon_v)_{SA}$ for a range of $(\epsilon_v)_{SA}$ up to 0.03% were used, depicted in Figs. 20(a) and 20(b). These relationships were obtained by referring to the relationship between E_{eq}/E_v and $(\epsilon_v)_{SA}$ obtained from similar cyclic triaxial tests on Toyoura sand presented in Fig. 21. The values of E_{eq} at $(\epsilon_v)_{SA} = 0.03\%$ from the cyclic triaxial tests, $(E_{eq})_{TXT}$, as obtained as above were plotted in Figs. 18(b) and 19 (i.e., hollow circles). It may be seen that the agreement in the E_{eq} value between the triaxial tests and the model tests was very good for Toyoura sand, while there was a large difference with the gravel. It is likely that this difference is due partly to the effects of interaction between the reinforcement and the gravel backfill.

Based on the above, it was decided to estimate the E_v values of backfill to be used in Eq. (3) by correcting the

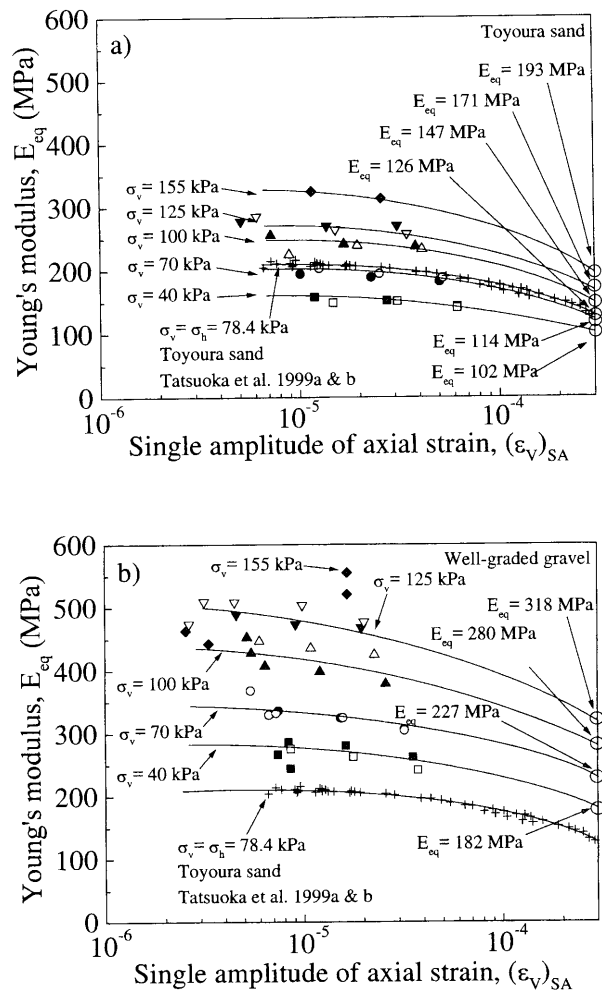


Fig. 20. Relationships between the equivalent Young's modulus E_{eq} and the single amplitude axial strain from cyclic triaxial tests at a constant σ_h on specimens at anisotropic stress states ($\sigma_v = 40, 70, 100, 125$ and 155 kPa and $\sigma_h = 40$ kPa) and the E_{eq} values estimated at $(\epsilon_v)_{SA} = 0.0003$ (0.03%): a) Toyoura sand and b) well-graded gravel

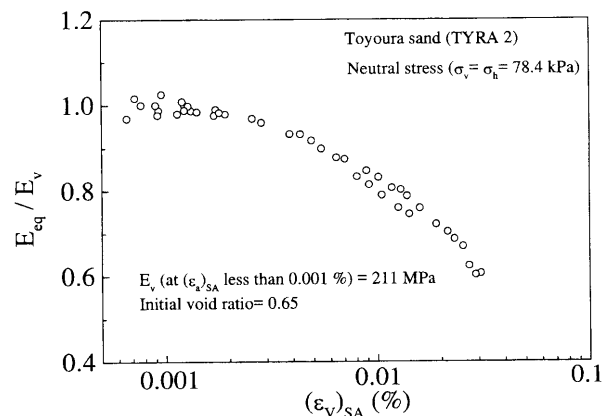


Fig. 21. Relationships between the equivalent Young's modulus E_{eq} and $(\epsilon_v)_{SA}$ from cyclic triaxial tests at a constant σ_h on anisotropically consolidated specimens of air-pluviated Toyoura sand ($\sigma_v = \sigma_h = 78.4$ kPa) (Tatsuoka et al., 1999a and b)

measured E_{eq} values for backfill (as represented by Eq. (4)) for the effects of strain-non-linearity, which are not based on the E_v values obtained from cyclic triaxial tests on the backfill materials (as represented by Eq. (1)). That is, the elastic Young's modulus E_v of the respective reinforced backfill was obtained as:

$$E_v = E_{eq} / [(E_{eq}/E_v) \text{ at } (\varepsilon_v)_{SA}] \quad (5)$$

where E_{eq} is the equivalent Young's modulus of backfill measured in each model test; $(\varepsilon_v)_{SA}$ is the single amplitude average vertical strain at which the E_{eq} value was obtained; and $[(E_{eq}/E_v) \text{ at } (\varepsilon_v)_{SA}]$ is the value of (E_{eq}/E_v) obtained by substituting the $(\varepsilon_v)_{SA}$ value into the relationship presented in Fig. 21 (this relationship is also plotted in Figs. 20(a) and (b)). The values of E_v thus obtained are plotted against σ_v in Figs. 18(b) and 19 (i.e., hollow squares), which were fitted by:

$$E_v = A^* \cdot \left(\frac{\sigma_v}{p_a} \right)^m \quad (6)$$

The best fitted values of the parameters are; $A^* = 208,651$ and $m = 0.25$ for Toyoura sand (Fig. 18(b)); and $A = 177,740$ and $m = 0.21$ for the well-graded gravel (Fig. 19). Equation (6) was used to obtain the values of E_v in Eq. (3).

Likely Effects of Swelling on the Residual Strain by Cyclic Loading

The values of $(\varepsilon^p)_{swelling}$ that were obtained as "measured $\varepsilon_{unloading}$ " minus " $(\varepsilon^e)_{unloading}$ from Eq. (3)" are plotted against in the unloading ratio UR in Figs. 11 and 12. It may be seen that for both the types of backfill soil, the increase in the residual strain ε_r as the unloading ratio UR increases from about 0.5 to 1.0 is well in accordance with the trend of $(\varepsilon^p)_{swelling}$. This is particularly the case with Toyoura sand (Fig. 11(a)). These results suggest that the residual strain of reinforced backfill by cyclic loading may not become very small when unloaded from the preload level to a very low or zero prestress level due to the following mechanisms:

- 1) the backfill may rebound irreversibly in the vertical direction upon unloading of vertical stress, which may damage the micro-structure of backfill; and
- 2) the elastic stiffness of the backfill against axial loading decreases with a decrease in vertical stress associated with unloading.

Based on the test results presented above, the following preloading and prestressing procedure can be recommended for prototype GRS structures;

- 1) The reinforced backfill is vertically pre-loaded to such a high load level that the difference between the preload and prestress levels is larger by a factor of two or more compared with the magnitude of design load (dead plus live).
- 2) Large irreversible (or plastic) strains should be generated by, for example, pre-cyclic loading at the preloading state.
- 3) The vertical load is decreased to a prescribed pre-

stress level such that the swelling of backfill is maintained at a small value. The results from the present study indicate that about a half of the preload level is relevant. After having reached the prestress level, the time-dependent swelling of backfill is not allowed to take place before the structure is opened to service.

- 4) Tie rods having a suitable stiffness are used to avoid a large decrease in the prestress due to residual compressive strain of backfill.

For a high seismic stability of reinforced soil structures, it is essential to keep sufficiently high prestress during seismic loading (Shinoda et al., 2001b). That is, the purposes of preloading and prestressing procedure are not only to minimize residual compression by long-term cyclic load (such as traffic load), but also to reduce the dynamic and residual deformations produced by high-level seismic load of the reinforced backfill.

CONCLUSIONS

The following conclusions can be derived from the results from the experiments and analyses presented above:

- 1) Residual vertical compression of reinforced soil backfill by vertical cyclic loading became substantially smaller by applying a relevant vertical preload to the backfill.
- 2) When the preload was fully unloaded before cyclic loading, the residual compression of backfill became very large. It is very likely that this trend was partly due to a relative large plastic swelling that took place by a release of elastic energy stored in the backfill and reinforcement, and partly to a decrease in the elastic stiffness of the backfill associated with a large decrease in vertical stress.
- 3) Within the limit of test conditions employed in the present study, the smallest residual compression of backfill was obtained when the prestress level at which cyclic load was applied was about a half of the preload level and the difference in the vertical stress between the preload and prestress levels was more than three times the amplitude of cyclic stresses.
- 4) The residual compression of backfill could be further reduced by allowing plastic strains to take place by applying cyclic vertical stresses at the preloading state.
- 5) The residual compression during cyclic loading of backfill decreased by using tie rods to maintain the prestress during cyclic loading. Despite the above, the tie rod tension decreased with development of residual compression of the backfill. Therefore, to keep the prestress high enough for a long duration, the residual compression of backfill should be minimised and some measures (such as using tie rods having a low stiffness) should be taken so that the tie rod tension does not decrease largely even when some residual compression of backfill takes place.
- 6) The residual compression of the backfill was smaller when the backfill was constructed using a well-graded

gravel reinforced with a metal grid compared with using a poorly-graded fine sand, Toyoura sand, reinforced with a polymer grid, for the same relative density of backfill.

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