Recent Case Histories of Permanent Geosynthetic-Reinforced Soil Retaining Walls, Tatsuoka & Leshchinsky (eds)
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Geosynthetic-reinforced retaining walls for bullet train yard in Nagoya

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ABSTRACT: The geosynthetic-reinforced retaining wall system (GRS-RW system), using a delayed cast-in-place concrete rigid facing, is described. The wall was used to reconstruct the existing slope of an embankment for a bullet train yard in Nagoya City. The total length of the wall is 930 m with an average wall height of 5 m. The post-construction performance of the GRS-RW bridge abutment is detailed.

1. INTRODUCTION

The construction of the geosynthetic-reinforced soil retaining wall system (the GRS-RW system) for expanding the area of the yard for the bullet train (Shinkan-Sen) at Hibitsu in Nagoya City is described in Tatsuoka et al. (1992). In this project, the advantages and disadvantages of the GRS-RW system, Terre Armee technique and the conventional RC cantilever wall were compared, and finally the GRS-RW system was adopted.

The existing slope of embankment for a total length of 930 m, with an average height of 5 m, was reconstructed to near vertical retaining walls. The construction was started in 1990 and completed in 1991.

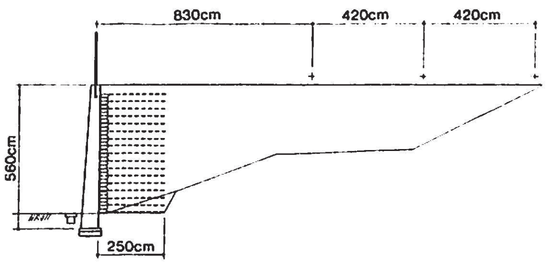

Fig. 1 Typical cross-section of GRS-RW at Hibitsu, Nagoya (Fig. 3 of Tatsuoka et al., 1992)

The successful design and construction, and very satisfactory post-construction behaviour of this GRS-RW system is an important milestone in the history of the GRS-RW system, leading to a subsequent wider use of this system.

In this report, first the completed walls are described. Subsequently, the post construction behaviour of the two GRS-RW bridge abutments is reported.

2. COMPLETED GRS RETAINING WALLS

Fig. 1 shows a typical cross-section of the GRS retaining wall of this project. **Plate 1** shows a typical view of the completed wall. Since most part of the wall face is next to public roads and close to nearby residential areas, it was made aesthetically pleasing by forming the appearance of natural stone using cast-in-place concrete together with special concrete framework.

Plate 2 shows a view of the crest immediately behind the facing of the completed wall. Many electric poles having their foundations located in the reinforced zone and a noise barrier fence on the top of the facing can be seen. As described in Tatsuoka et al. (1992) and Tatsuoka (1993), with the help of a full-height continuous rigid facing, these types of structure

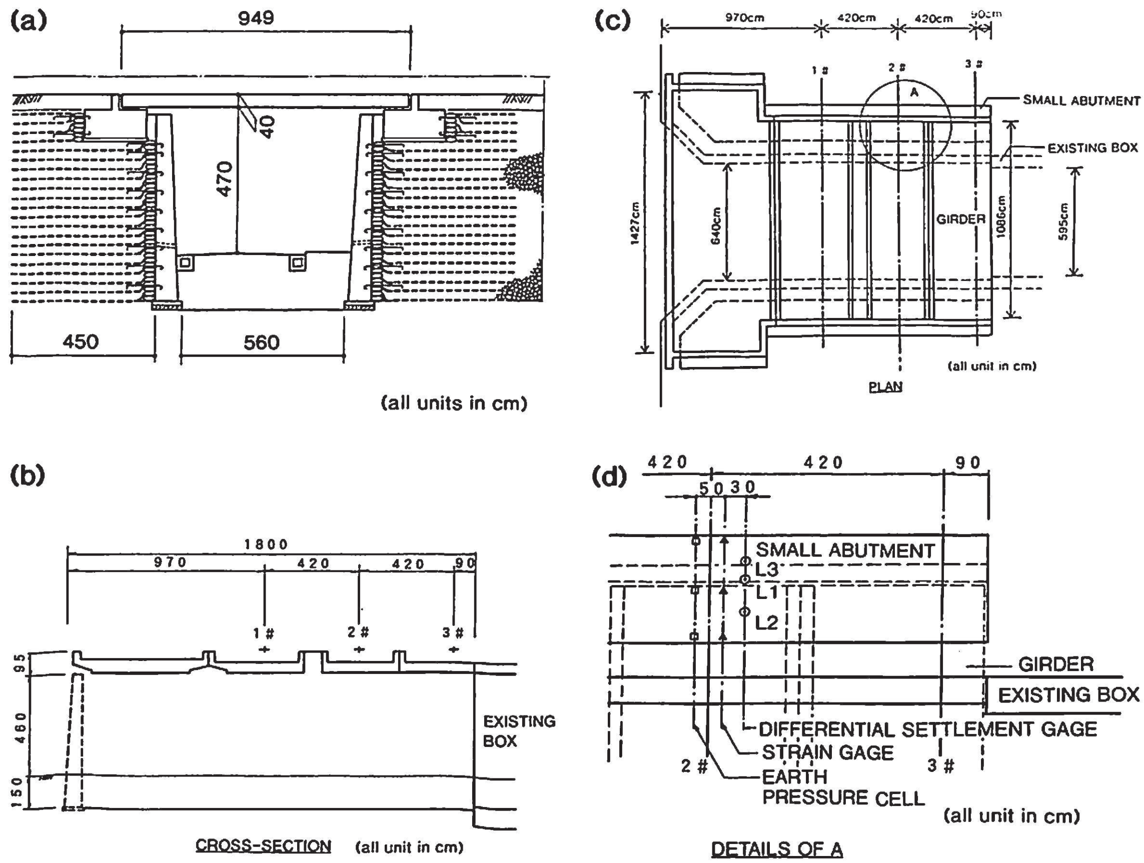

Fig. 2 GRS-RW bridge abutments at Hibitsu, Nagoya; (a) front view (Fig. 2 of Tatsuoka et al., 1992), (b) cross-section, (c) plan and (d) close up view of zone A in (c)

which may exert concentrated outward lateral and vertical load on the walls could be constructed without causing problems. Thus this factor is considered an important advantage over Terre Armee technique.

Fig. 2 shows two GRS-RW bridge abutments supporting a railway bridge girder. **Plate 3** shows the view after completion and **Plate 4** shows the view from above of the bridge supported by the GRS-RW bridge abutments. Considering its critical use, a relatively strong grid with a tensile rupture strength of 6 tonf/m was used. As backfill soil, a well-grained sandstone gravel (U_c= 91.8 and D_{50}= 4.8 mm) was used and was compacted very well to a dry density of as high as about 2.2 g/cm^3.

The stability analysis of the GRS-RW bridge abutments for lateral sliding and overturning was performed by limit-equilibrium methods, which are explained in detail in Horii et al. (1993) in this volume. In this analysis, a partial safety factor of 1.5 was used for the pullout failure of reinforcement. The calculated safety factors are plotted in Fig. 3 against the average vertical pressure from the bridge girder at the concrete block with a base of 2.0 m x 11.8 m on the crest of the abutment. It may be seen that the safety factor for the design train load of 2 tonf/m^2 is larger than six. In the static loading test performed immediately after the completion of the wall construction, a total pressure of 40 tonf and an average pressure of 2 tonf/m^2, which is similar to the design train load, was applied to the concrete block on one of the GRS-RW bridge abutments (the left-side in Fig. 2a). The settlement was only 0.1 mm. The results of

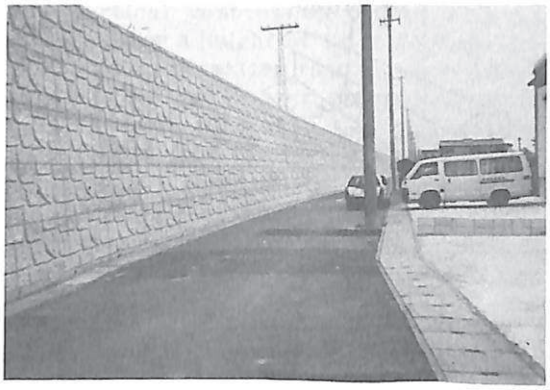

Plate 1 View of the completed GRS-RW at Hibitsu, Nagoya

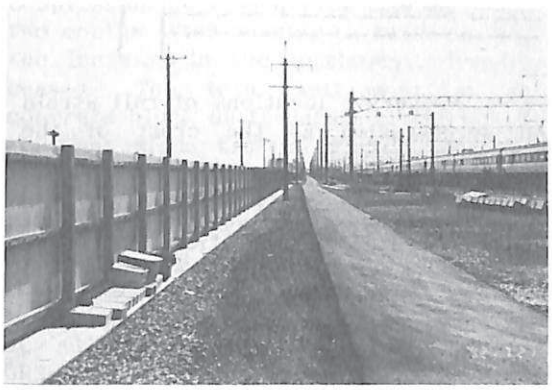

Plate 2 View of the crest of the completed GRS-RW at Hibitsu, Nagoya

the static loading tests and its FEM simulation are reported in Tateyama and Murata (1994).

3. BEHAVIOUR OF GRS-RW BRIDGE ABUTMENTS

3.1 Measuring system

Trains started passing on the bridge supported by the GRS-RW bridge abutments on 29th October 1992. The long-term behaviour of the abutment on which the above-mentioned static loading tests were performed has been monitored continuously since the completion of the wall construction.

Plate 3 View of the completed GRS-RW bridge abutment at Hibitsu, Nagoya

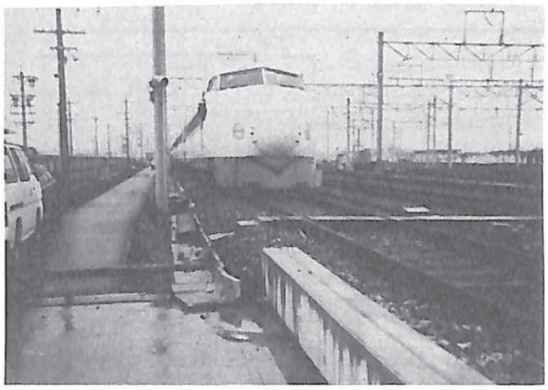

Plate 4 View of the bridge supported by the GRS-RW bridge abutments at Hibitsu, Nagoya

As shown in **Fig. 4**, precipitation, strains in the reinforcement, vertical earth pressure, pore water pressure and settlement in the backfill, strains in the steel reinforcement in the rigid facing and its tilting at the wall face were measured. The locations of the settlement gauges, the reinforcement strain gauges and the earth pressure gauges are also indicated in Fig. 2(d). The data was automatically acquired every one hour by the system shown in **Fig. 5**.

The dynamic behaviour of the abutment during train passing was also recorded. In addition to those shown in Fig. 4, the rail strain and the acceleration at the crest of the abutment were recorded by means of a

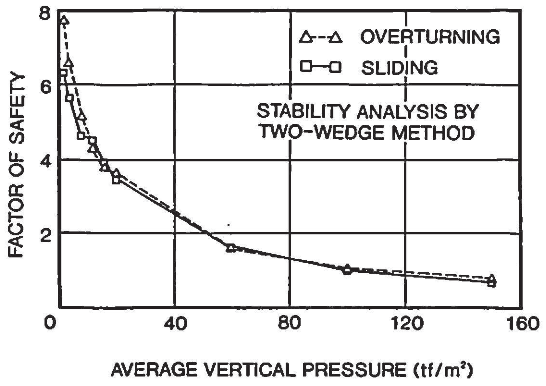

Fig. 3 Results of the stability analysis of the GRS-RW bridge abutments at Hibitsu, Nagoya

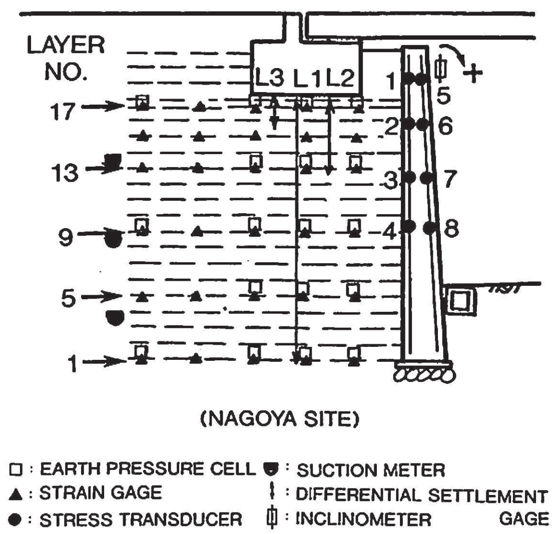

Fig. 4 Arrangement of measuring instrumentation in the GRS-RW bridge abutment at Hibitsu, Nagoya

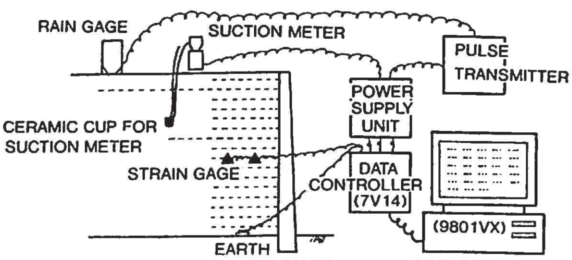

Fig. 5 Recording system for the GRS-RW bridge abutment at Hibitsu, Nagoya

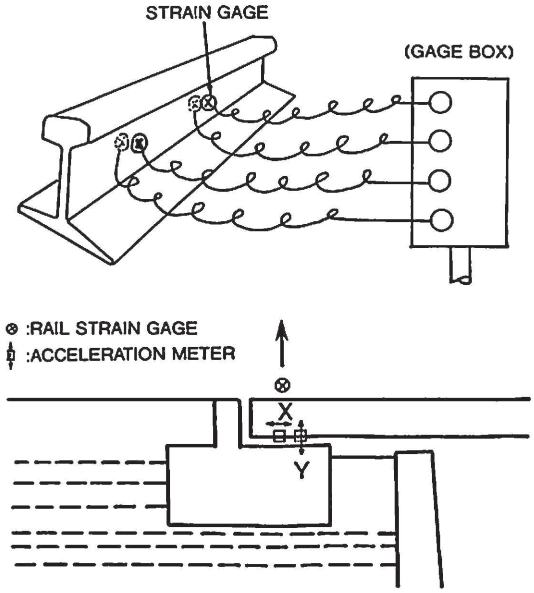

Fig. 6 Measuring locations of rail strain and acceleration at the crest of the abutment

data recorder. The locations of these measurements are indicated in **Fig. 6**.

3.2 Long-term behaviour

Fig. 7 shows the time histories of some typical measurements for 1.5 years from September 1991 until February 1993. The measured quantities shown in Fig. 7 (except for those shown in **Fig. 7a**) are the increments defined as zero at the completion of construction in September 1992. API is the antecedent precipitation index, which represents the humidity condition. API at time t is defined as $P_o + K\cdot P_1 + K^2\cdot P_2\ K^3\cdot P_3 \cdots + K^n\cdot P_n$, in which K is a constant (= 0.8 in this case), P_o is the rainfall for one day preceding the time t and P_n is the daily rainfall on the n^{th} day before time t.

The vertical earth pressures measured near the wall base and immediately below the concrete block (**Fig. 7b**) show a similar trend (but in different magnitudes) to the placing and removing of the temporary bridge girder and the placing of the permanent one. However, the long-term change with train passing is not noticeable.

The vertical compression of the backfill measured for a height of 1.5 m (L2) near the facing is much smaller than those measured for shorter and longer gauge lengths of 0.6 m (L3) and 5.0 m (L1), i.e., at locations more remote from the facing (**Fig. 7c**). This result shows that the concrete block has rotated with larger settlement at the heal than at the toe. However, the amount of settlement and rotation is very small.

The general trend is that the compression of the backfill increased corresponding to the precipitation, probably due to the reduction in suction by rainfall. The settlement due to dynamic train loads after 29th October 1992 can also be noticed. However, the rate of the vertical compression decreased with time and about two months after opening to train passing, the increase in the settlement virtually ceased. The total settlement at the concrete block on the crest of the GRS-RW abutment since traffic started, measured independently, is 2.0 mm. This value is very small and well in accordance with those shown in Fig. 7(c). Although the settlements at the track of the GRS-RW abutments was not negligible, it was not larger than those measured on the tracks at adjacent locations in the ordinary GRS-RW sections. Any noticeable unequal settlement along the track on and adjacent to the GRS-RW bridge abutment has not been reported.

As seen from **Fig. 7(d)**, the response of the reinforcement tensile force to the placing and removing of girder and train passing is much less noticeable than that of the vertical earth pressure. Because of some unknown reasons, the reinforcement force at E16 (at the remotest measuring point from the facing) was very large. Considering that all the other measurements did not show such a large value, this value is not very reliable (yet the maximum tensile force as measured is only about 1/6 of the tensile rupture strength). In any case, the increase in the tensile reinforcement force nearly ceased about two months after opening to train passing.

Fig. 7(e) shows the inclination of the rigid facing (positive for outward inclination). The change of the inclination during this term of measurement is negligible. It is readily seen from Fig. **7(f)** that the strains in the steel reinforcement in the facing followed seasonal temperature changes. This made possible changes due to external loading difficult to be detected. Despite the above, the following behavioral trend can be seen. When the bridge girder was placed on the wall crest, gauges S1 ~ S4 located near the back of facing exhibited tensile strains, whereas S5 ~ S8 near the wall face exhibited compressive strains. This means that the facing was bent outwards (i.e., the top of facing tilted outwards). Through the period of measurement, in general, the tensile strains are larger in the gauges near the back of facing (S1 ~ S4). These results suggest that the overall bending moment was activated in the facing to resist the earth pressure thrust on the back face of facing. Similar to other measurements, the strains in the steel reinforcement became very stable about two months after opening to train passing.

In summary, the measurements shown above indicate stable behaviour of the GRS-RW bridge abutments.

3.3 Dynamic behaviour

On October 29 1992, the first train of Shinkan-Sen passed above the GRS-RW abutment at a velocity of about 9 km/h. The wheel load was 16 tonf. The wheel configuration is shown in **Fig. 9**. **Fig. 8** shows the time histories of several typical measurements during train passage. These values are in increments caused by the train load. It may be seen from Fig. 8(b) that due to the relatively low velocity, the horizontal and vertical acceleration on the crest of the GRS-RW abutment were very low.

The correspondence between the vertical earth pressure and the reinforcement tensile strain shown in Figs. 8(d-1) and (d-2) is very obvious, but it does not apply to other cases in Fig. 8. The same tendency was observed among the corresponding cases during the static loading test. In any case, the change in the reinforcement force is very small with maximum

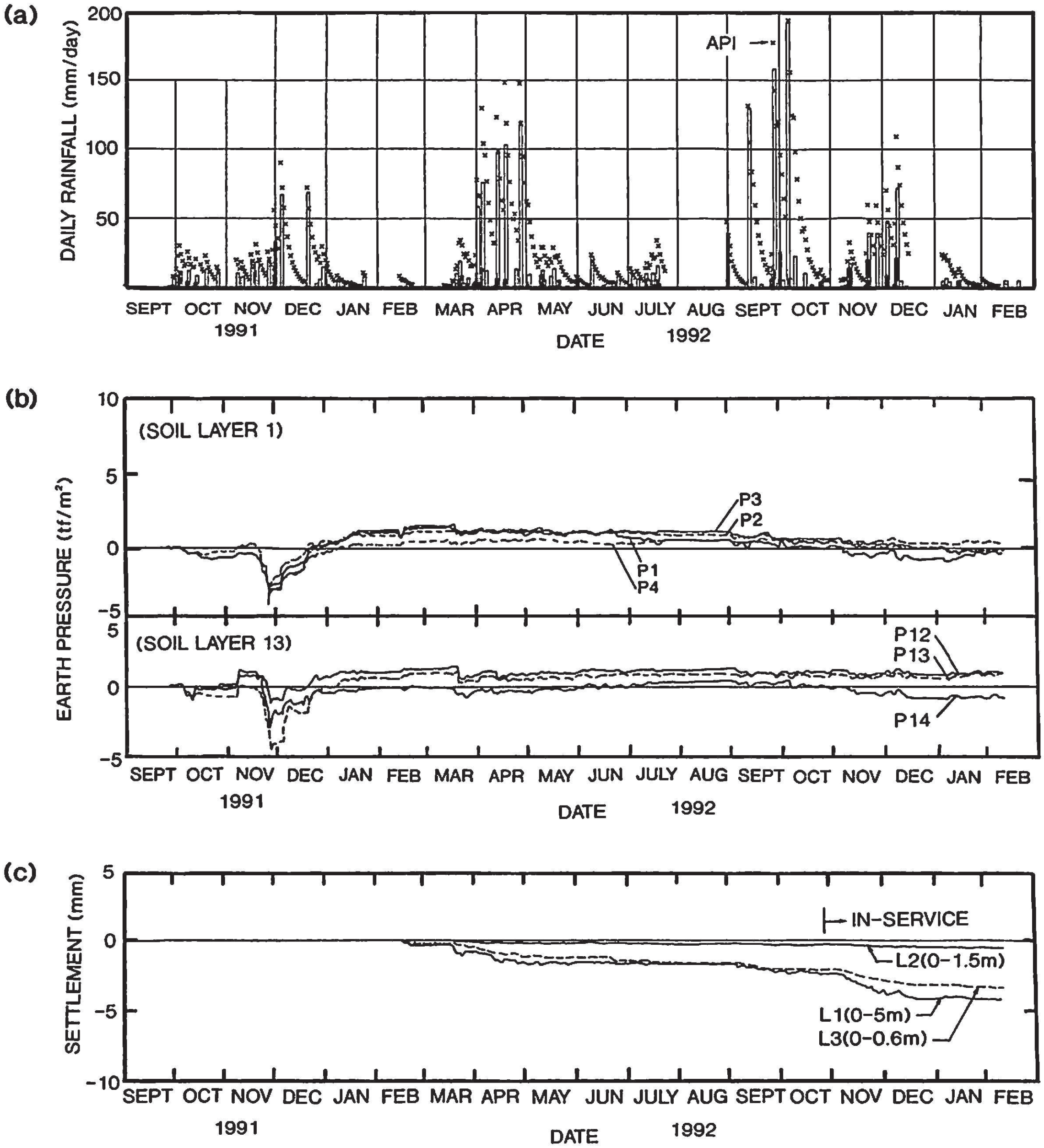

Fig. 7 (continue to next page) Time histories of typical measurements from September 1991 until February 1994 (defined as zero in September 1992): (a) Daily precipitation and API, (b) vertical earth pressure (soil layers No. 1 and 13), and (c) vertical compression at L1, L2 and L3

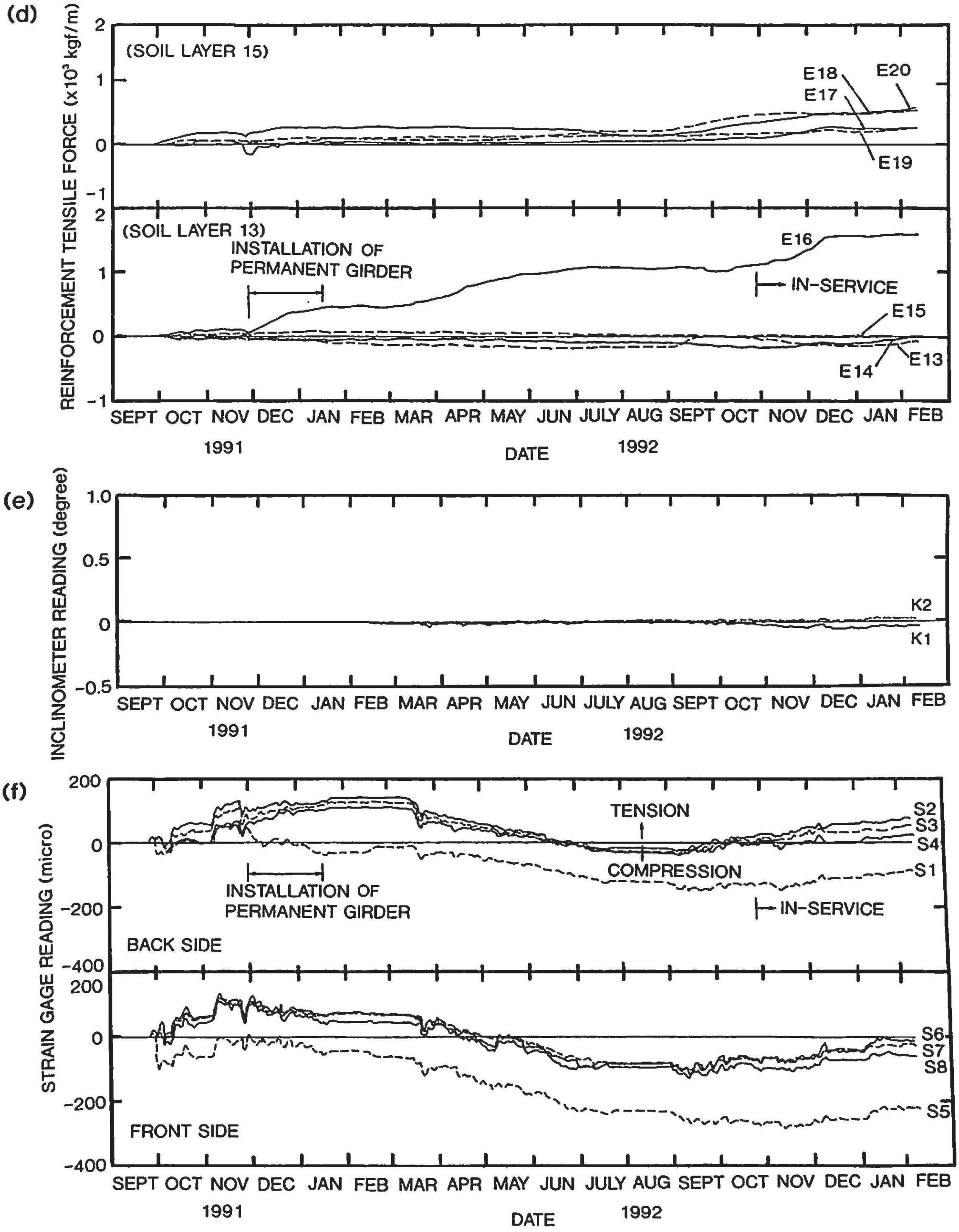

Fig. 7 (cont.) (d) reinforcement tensile force (soil layers No. 15 and No. 13), (e) inclination of the wall face of rigid facing, and (f) strains in the steel reinforcement in the facing numbered from the top (see Fig. 4 for the soil layer number. In each soil layer, the instrumentation located immediately behind the back face of facing to more distant locations is numbered sequentially)

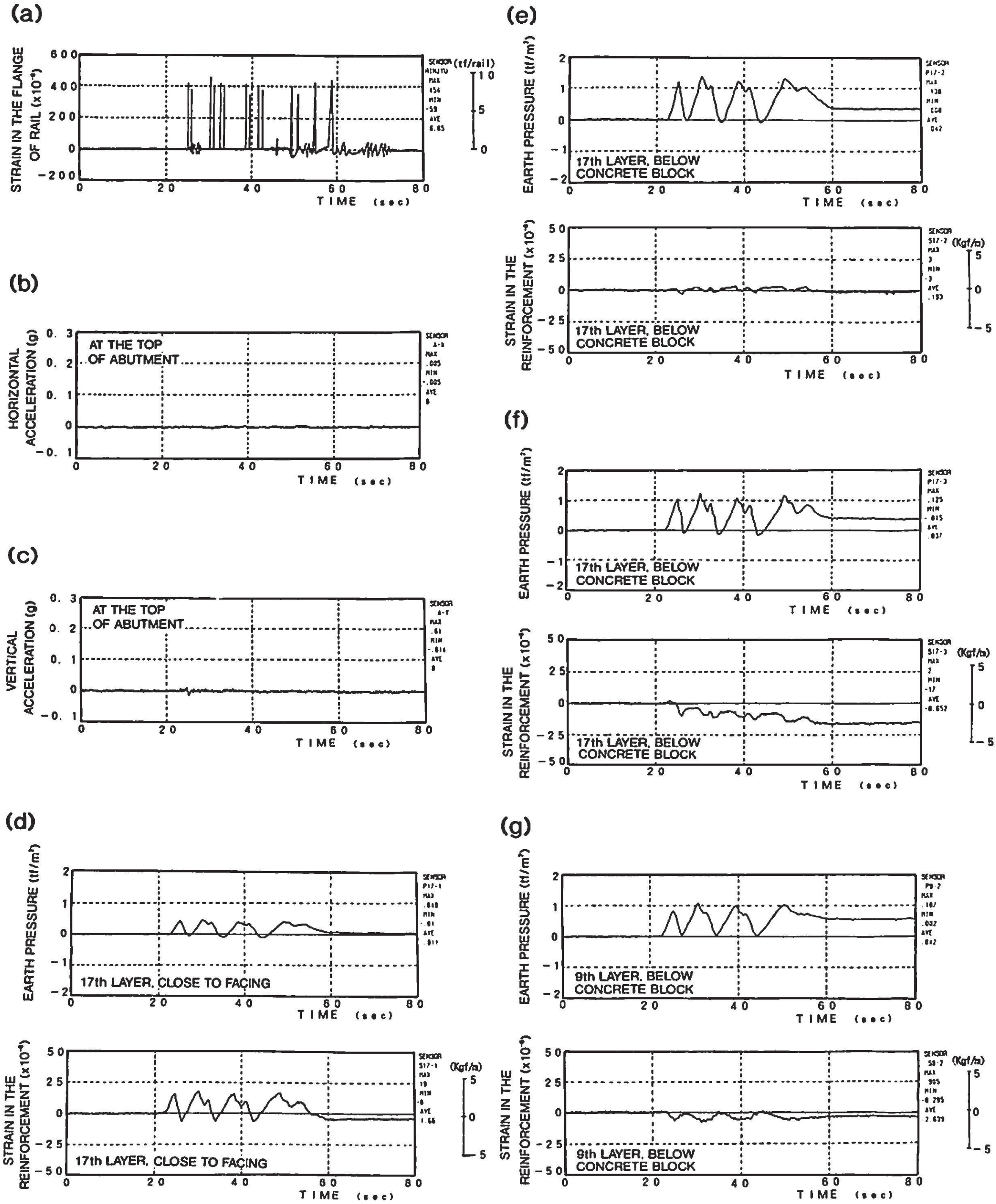

Fig. 8 Time histories during the first passing of train on October 29 1992 of the changes in: (a) strain in the flange of rail, (b) and (c) horizontal and vertical acceleration on the crest of the GRS-RW abutment, (d-1) and (d-2) ~ (f-1) and (f-2) pairs of vertical earth pressure and reinforcement tensile force in the soil layer No. 17, (g-1) and (g-2) a pair of vertical earth pressure and reinforcement tensile force in the soil layer No. 17 (see Figs. 4 and 6 for the locations of measuring instrumentation)

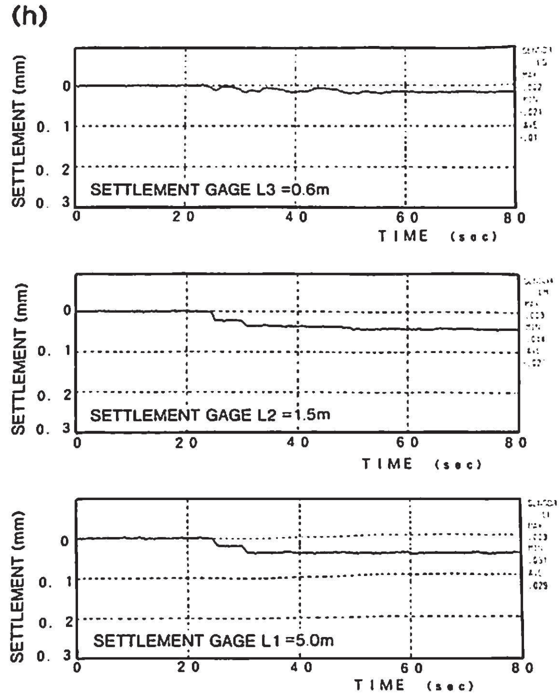

Fig. 8 (cont.) Time histories during the first passing of train on October 29 1992 of the changes in: (h-1) ~ (h-3) vertical compression in the backfill below the concrete block at L1, L2 and L3 (gauge lengths= 5m, 1.5 m and 0.6m) (see Figs. 4 and 6 for the locations of measuring instrumentation)

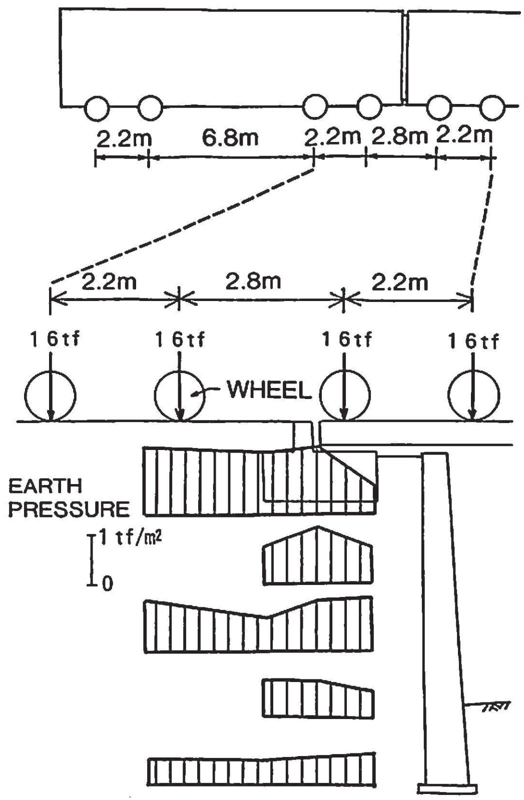

Fig. 9 Typical record of earth pressure when a train stopped on the GRS-RW abutment

tensile force of only 5 kgf/m.

It may be seen from Figs. 8(d-1), (e-1) and (f-1) that the maximum earth pressure at the measuring point closest to the facing in the soil layer No. 17 is smaller than those measured at more remote points. This behaviour was also observed when a train stopped on the abutment as shown in Fig. 9. This result may suggest that although it is sufficiently large, the degree of restraint against outward lateral spreading of soil by tensile reinforcement with the help of a rigid facing was not as large as it is at level ground of reinforced dense gravel. Yet, the degree of this non-uniform earth pressure was very small and did not lead to any problem.

It may also be seen from Fig. 9 that the increase in the vertical earth pressure at the wall base due to train load decreased to about half of that measured immediately below the concrete block. When this earth pressure is modelled by a uniform increase in the zone between the back face of the facing and a plane radiating from the heel of the concrete block, the distribution angle becomes 27° relative to the vertical.

It may be seen from Fig. 8(h) that the vertical compressional strains in the backfill soil below the concrete block is very small. The settlement of the concrete block as measured at L2 was about 0.05 mm, which is extremely small; this value is much smaller than the 5 mm allowable maximum settlement of the track during train passing. This value is slightly smaller than that observed during the static loading test (= 0.1 mm). The ratio of the maximum settlement at L2 to the maximum earth pressure measured in soil layer No. 17 below the center of the concrete block during this first train passing (Fig. 8(e-1)) was 0.05 mm/1.3 tonf/m^2, slightly smaller than that during the static loading test (0.06 mm/1.1 tonf/m^2). This is due to the effect of some densification and/or cyclic prestraining at the small strain stiffness range of the gravel layer.

The vertical compressional strain by this first train passage averaged for each gauge length ranged between about 1.0 and 2.7 x 10^{-5}. Although these values are very small, they are plastic and can accumulate. This is in accordance with the gradual increase in the settlement during the first two months after the opening to train passing as shown in Fig. 7(c).

4. CONCLUSIONS

The geosynthetic-reinforced soil retaining wall (GRS-RW) constructed for expanding the Shinkan-Sen yard at Hibitsu in Nagoya city was described. The behaviour of the GRS-RW bridge abutments observed for 1.5 years and the dynamic behaviour during the first passage of train were reported. Observations and measurements showed that the GRS-RW bridge abutments (and ordinary part of GRS retaining walls) have been very stable without showing noticeable deformation.

The successful design and construction and very satisfactory post-construction behaviour of the GRS-RW system at Hibitsu has enhanced the adoption of the GRS-RW system at other locations in Japan.

ACKNOWLEDGEMENTS

The authors appreciated the review of paper by Prof. D. Leshchinsky of the University of Delaware, U.S.A., and Dr. H.I. Ling of the University of Tokyo.

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