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# Three topical Terre Armée walls in Japan

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ABSTRACT: Since the invention of Terre Armée Method by Mr. Henri Vidal in 1963 and its subsequent development for use in civil engineering applications, including road, highway and railway constructions, for more than 4,000 walls or 1,300,000m² in wall area, have been constructed in Japan. Recently this Terre Armee Method has been used in the various kinds of projects in addition to those already popularized. We show some of them for your interest. "TA" will be collectively used here to cover all constructive structures employed Terre Armee Method.: 1. Tiered TA, 2. In waterside, and 3. Abutment by TA.

## 1 EXAMPLE OF TIERED TA

### 1.1 Preface

Based on numerous experiences in designing of TA, we attempted on new application so called Tiered TA, without having specific knowledge of it, and were successful. We released a paper concerning design method of the Tiered TA and introduced this project.

### 1.2 The Design Method of the Tiered TA

The impotant points in the design method of the Tiered TA are studies on the external and internal stabilities.

(1) Study on the External Stability of Tiered TA

As the Tiered TA is often employed in big construction works, we should check circular slip of the total banking and each TA as shown in Fig.1. If the base ground is poor, we should check the displacement in addition to the circular slip.

If the stability of the circular slip is not enough, it is preferred to lay longer strips or modify the banking materials.

In particular, it is very effective for improving stability of the circular slip to use longer strips in the lower strata. In addition, improvement of the base ground is effective to prevent from displacement.

(2) Study on the Internal Stability of Tiered TA

In the study on the internal stability of Tiered TA, we must consider the following 3cases.

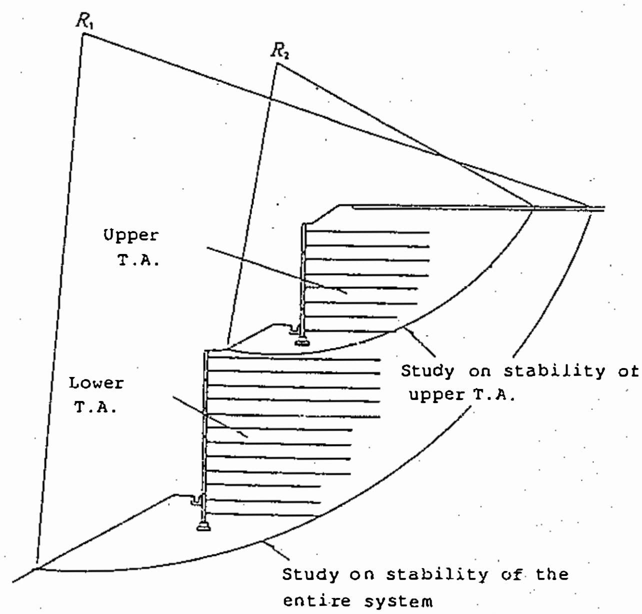



Fig.1.1 Study on the External Stability

i ) Referring to Fig.2(a), upper TA resides very close to lower TA, within the active state area of lower TA. In this case, it is assumed that upper TA greatly affects lower TA.

ii ) In Fig.2(b), upper TA is located beind the active state area of lower TA but within the distance of summation of the installation height difference between upper TA and lower TA and is as large as virtual wall height of lower TA by an amount equal to 0.4 times of it. In this case, it is assumed that upper TA affects lower TA.

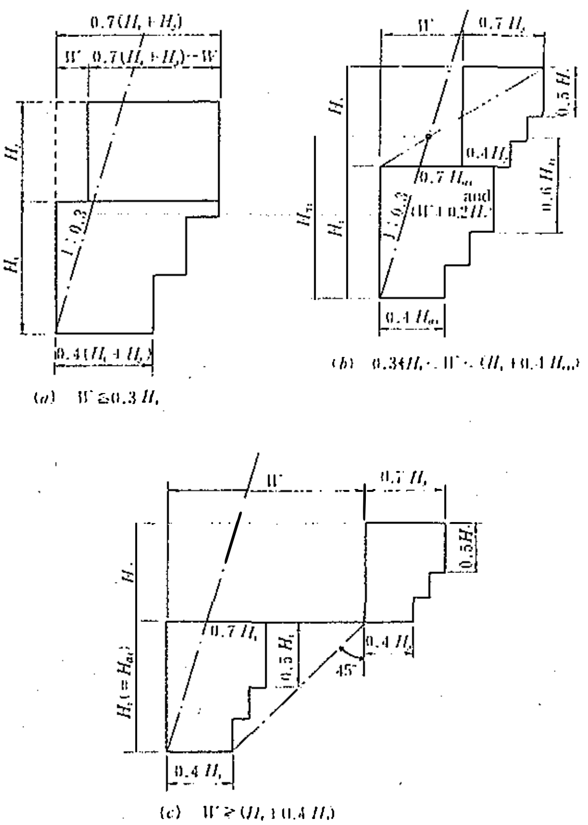


Fig.1.2 Positional Relatioship of Tiered TA and Minimum Length of Strips

iii) Upper TA is considerably apart from lower TA as shown in Fig.2(c) (where upper TA and lower TA are farther than distance of ii). At that time, it is assumed that upper TA will not affect lower TA by no means.

We should study the internal stability of these 3 cases with the following assumptions.

In case of i), the upper and lower TA shall be regarded as one single TA. In other words, it is assumed that the wall surface of upper TA is located at the same position as the wall surface of lower TA. Therefore, the layout of strips and the length of them are determined as one single TA and then only the position of the wall surface of upper TA is set back to the position planned Consequently, the length of strips in upper TA becomes smaller by amount of set-back of upper TA from one single TA (see Fig. 2(a)).

In the case of ii), each of upper and lower TA is regarded as an independent one. However, lower TA is assumed TA with a virtual slope of having banking volume equal to that of the total banking including upper TA In addition, upper TA too has to be studied as an independent TA, as if TA is situated on ordinary banking.

As a result of the study described above when the strips of lower TA are short and upper TA is not loaded on the strips of lower TA, the sliding of lower TA is induced by the load of upper TA. Therefore the length of strips in the upper strata of lower TA (more specifically, as many layers as 1/3 of the wall height from the top of lower TA) shall be made longer than the length reaching the center point of of the strip length in the lower part of upper TA (see Fig.2(b)).

In the case of iii), it is expected that upper TA will not affect lower TA, so the stability is studied assuming that each TA is independent (see Fig.2(c)).

## 1.3 The Construction of Tiered TA

A Tiered TA was constructed as a banking of a golf driving range. Height of each TA is 15.0m (lower TA ) and 10.5m(middle and upper TA ) respectively as shown in Fig.3. This project was the the largest scale construction in Japan as far as wall height is concerned. The banking materials were sands and crushed stones, both were taken at the job site.

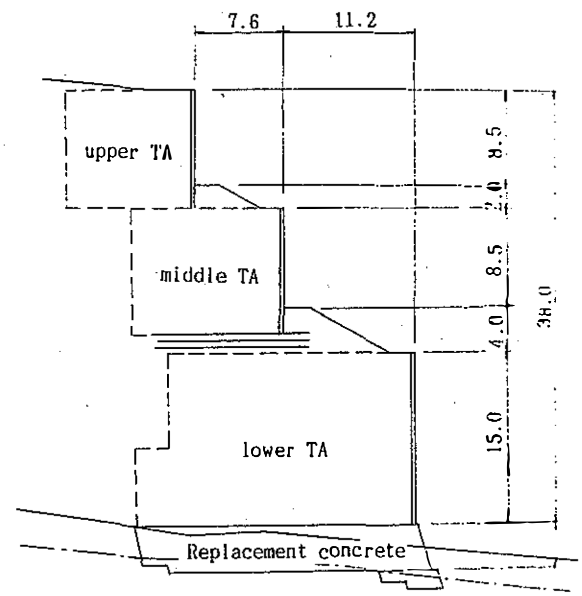



Fig.1.3 The Cross Section of This Project

This project was studied in accordance with principles as described above.

Basically, it was intended to utilize space effectively and as wide as possible while achieving the most economical cross section. For this purpose, upper TA was not set on the active state area of lower TA. Usually, the safety factor of the circular slip is taken at Fs=1.25 to study external stability, however, the safety factor for this project was taken at Fs=1.5 considering the scale of the project, the largest ever.

Resultingly, the length of strips in lower and middle banks' TA was made very long.

### 1.4 The Construction Works

The construction works required atmost care for absolute safety, since a high banking ever had been planned on a large-scale. And then the displacement of wall surface and the sinking of TA were measured and compared with calculated values. The result was very good as expected. While, the displacement of wall surface of lower TA was within 10cm meeting with calculated values, after upper TA was constructed.

## 2 CASE EXAMPLE OF DESIGN AND CONSTRUCTION FOR TA IN WATERSIDE

### 2.1 Preface

TA in waterside means a TA structure constructed directly abutting on waterside of a river, a lake, a pond or seacoast.

The case, as presented in this report, a TA was constructed in Hiroshima Pref. that marked a forerunner of waterside TA in Japan

### 2.2 Summary of Design

Designing a TA in waterside poses a vital subject as to how to evaluate the coefficient of friction between the underwater backfill materials and the strips, and the residual water level resulted from a water level differential between front of the wall and the backfill.

The focus placed on this subject has underlined the method and relevant considerations having been given for designing the said work.

(1) Coefficient of Friction and Residual Water Level

We conducted a test on the backfill as to its shearing force on both saturation and partial saturation conditions, with the result as shown in Tab.2.2.

Next, we employed a non-stationary calculation method, on the basis of the FEM analysis program for the two dimensional unsaturated seepage flow, to find out a possible relationship between a water level at front of the wall and the one inside the backfill, with result shown in the Tab.2.2.

Tab.2.1 and 2.2 determined that the design should use a crusher-run with a content of fines less than 5% passing the 75 $\mu$m and permiability coefficient more than $1\times10^{-3}$cm/s.

In other words, our design was based on an underwater friction coefficient same as the one in partial saturation condition, and on a residual water level deemed anyway not taking place.

Tab.2.1 Coefficient of Friction & Angle of Internal Friction in Underwater

| Persentage of fines | Coefficient of friction | Internal friction angle |
|---|---|---|
| Less than 15% | 1.5 ～ tan36° | More than 35° |
| 15～25% | 1.5 × tan25/36 ～ tan25° | More than 35° |

Tab.2.2 Residual Water Level

| Permiability co-efficient of backfill | Grade of residual water level ($\Delta$h) |
|---|---|
| $k \geqq 1\times10^{-3}$cm/s | In case of $H \leqq 10$m v=1m/day, $\Delta h=0$ |
| $10^{-4} \leqq k < 10^{-3}$cm/s | $\Delta h > 0$ because of H,V |
| $k > 10^{-4}$cm/s | $\Delta h > 0$ at all time |

note) H; volume of drawdown at front of the wall (m)
v; speed of drawdown at front of the wall (m/day)

(2) Relevant Considerations

In addition to the designing method used, as explained in the foregoing subparagraph (1), the following points were taken into consideration in the course of designing the work.

① Facing Material

Concrete skins are directly exposed to force from the outside. Drifting materials could hit the wall. Thus, more strengthened skins are required. As a countermeasure, we made each skin thicker than usual, from 18cm to 22cm, and increased number of reinforcing bars inside skin by 2.8 times more than usually put in.

② Strip

We adopted Cm=2.0mm, 1.0mm more than that of commonly adapted, for designing this work, on the basis of some data obtained from France, because strips underwater can be deemed quick in corrosion compared with those not underwater.

③ Prevention of Backfill Erosion

For preventing the backfill from being caused to erode by drawdown at front of the wall, we applied filter materials (geotextiles) to the joints horizontally, in addition to normal filtering to vertical application, from backfill side.

④ Prevention of Scour

Underwater TA should make it fundamental design and construction basis to make the foundation deep enough not to be scoured.

In this work, we made a substitutive concrete foundation(3.5m high) in direct contact with floor rocks under water before placing the bottom of TA structure and backfilled by 4m in depth to waterside as deemed effective in preventing scouring.

The typical cross section made on the bases of subparagraph (1) and (2) above is indicated in Fig.2.1 below.

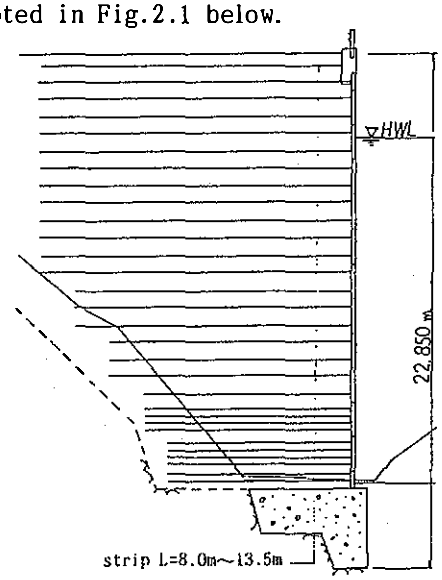



Fig.2.1 Typical Cross Section

2.3 Summary of Construction

The construction method for TA in waterside can be divided into two, one is for under water work and the other for dry work. We applied the latter by erecting temporary sluice bank wall utilizing soil excavated there. We employed a ratio a ratio isotope measuring instrument during construction work to monitor the degree of compaction and the water content at the same time, to put the backfill compacttion under good control.

Also we conducted a series of pull-out tests and in-situ permeability test, in order to ascertain the coefficient of friction between the backfill and the strips. The measures we thus took during the construction proved to be such as being summarized below.

(1) Controlling Compaction of Backfill

TA makes it the basis to compact the backfill more than 90% satisfying the maximum dry density as stipulated by JIS A 1210 A and B Methods (2.5kgf rammer).

In this work, we controlled the compaction according to the standards as shown in Tab.2.3, based on the result of the precedent experimental banking of the backfill. Also we monitored and processed the data of what we obtained about the degree of compaction and the natural water content, hoping that it will prove to be an instruction to a wide field of TA in waterside.

Fig.2.2 and 2.3 show all the result we got in this way.

Tab.2.3 Standards for Controlling Soil Compaction

| | |
|---|---|
| Standard lift thickness | 25cm |
| Type of rollers | Vibrating type 4tf |
| Frequency of rolling | 8 times |
| Frequency of control | Min.10 locations per each unit of rolling |
| Subjects to control | Degree of compaction & water content |
| Control method | Ratio isotope measuring |

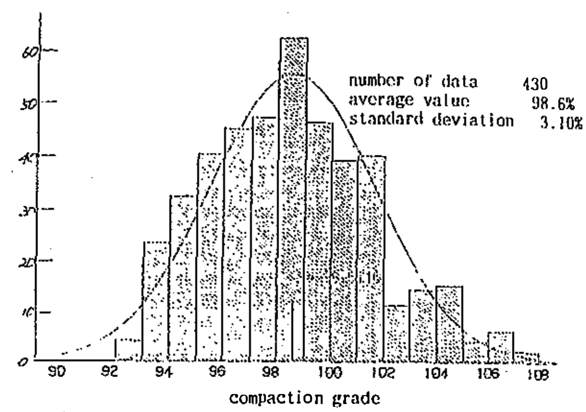



Fig.2.2 Distribution of Compaction Grade

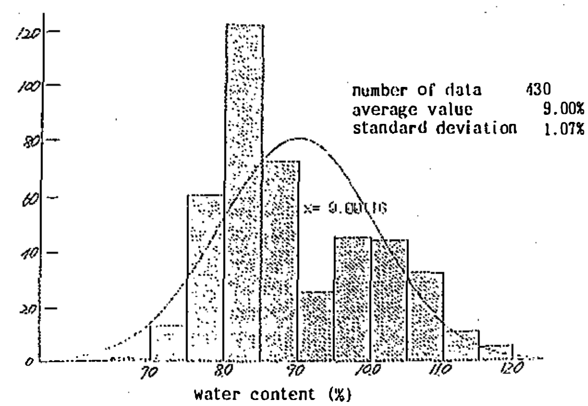



Fig.2.3 Distribution of Water Content

(2) Pull-out Test on Strips

In this work, we started with ascertaining if the design coefficient is actually realized on the spot. We pulled out with a jack the test strips which had been buried in readiness to estimate the friction coefficient on the maximum pull-out force and to compare with the design value.

The attempt proved the actual coefficient far surpassing the one designed, as shown in Tab.2.4.

Tab.2.4 Measurement Result of Coefficient of Friction

| No. | Values measured | Design value | Measuring condition |
|---|---|---|---|
| 1 | 1.299 | 1.049 | Le=5.5m, h=3.50m |
| 2 | 2.020 | 1.146 | 4.5 2.75 |

note) Le; length of a strip inside soil
h ; thickness of soil covering a strip

(3) In-situ Permeability Test
The residual water level inside the TA backfill block to be caused by drawdown at front of the wall may tend to become lower if permeability coefficient of the backfill is larger.
We based our design on the assumed coefficient k= $1\times10^{-3}$cm/s, which was, however followed by in-situ permeability test to ascertain the actual figure. The tests proved the actual coefficient surpassing the design value, as shown in Tab.2.5.

Tab.2.5 Results In-situ Constant Head Permeability Test (cm/s)

| No. | Values measured | Average values | Design values |
|---|---|---|---|
| 1 | $1.5\times10^{-2}$ | | |
| 2 | $1.8\times10^{-2}$ | $2.6\times10^{-2}$ | min. $1\times10^{-2}$ |
| 3 | $4.5\times10^{-2}$ | | |

## 2.4 Conclusion

It is noteworthly that various tests conducted on the spot seeking the distribution of the degree of compaction, the coefficient of the friction and the local water permeability coefficient on the crusher-run used in this case like other cases of TA in watersides turned out the results far exceeding the original expectations.

# 3 MEASUREMENT OF DYNAMIC LOAD ON TA ABUTMENT

## 3.1 Preface

Mixed abutment with exterior support has been constructed for the first time in Japan in a form as shown in Fig.3.1.
Both bridge abutments are based on caisson foundation ($\phi$2.5 m).
The wall surface of the TA bank was isolated 30cm from the outer end of the girder rack while connecting both members using approach cushion. The structure of the approach cushion is shown in Fig. 3.2.
Dynamic loading tests by driving a heavy vehicle on the bridge were carried out. The test results are mentioned herein.

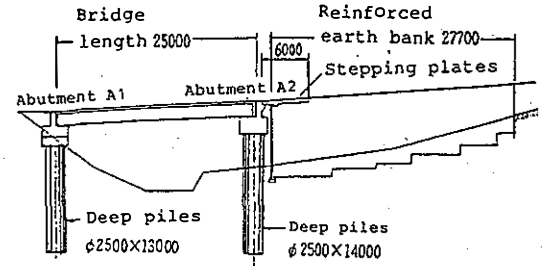



Fig.3.1 Standard Section of Shiomi Bridge

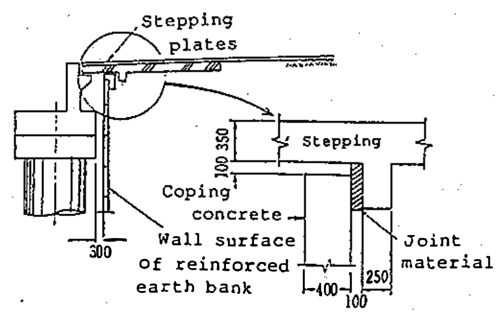



Fig.3.2 Structure of Approach Cushion

## 3.2 Dynamic Loading Measurement

Measurement items in dynamic loading included the change of stress in the embeded in the bank, vibration acceleration and displacement on the wall surface and the base of the abutments using accelerometers, when heavy vehicles ran.
(1) Case of the Experiment
All experimental cases of the dynamic load measurement are shown in Table. 3.1. The experiment was performed in 8 cases by varying the conditions of running loads and the modes of the accelerometers.
(2) Running Load
The 2 ×10-tf trucks shown in Tab.3.1 were operated as running load.
(3) Measurement Items and Locations
Measurement locations measured by the accelerometers are shown in Fig.3.3.

## 3.3 Measurement Results

(1) Vibration Accelerations on the Wall Surface Measured by the Accelerometers
Accelerometers were mounted on 3 locations, at the upper part (Spot A), the lower part (Spot B) on the wall surface

and the center part on the bridge surface of the abutment (Spot C).
The maximum values of the result measured with the accelerometers in dynamic loading are shown in the following.
At each measurement spot, 2-directional components in horizontal and vertical vibration were measured along the bridge.
Fig.3.4 and 3.5 show their distribution state.

Tab.3.1 List of Experimental Cases

| Case | Running load 20 tf/truck | Accelerometer measurement mode | Running speed km/h | Running time s/20 m | Remarks |
|---|---|---|---|---|---|
| 1-1 | A truck | Acceleration | 40 | 1.81 | |
| 1-2 | " | " | 42 | 1.71 | |
| 2-1 | A truck | Displacement | 40 | 1.79 | |
| 2-2 | " | " | 41 | 1.76 | |
| 3 | " | Acceleration | 19. | 3.75 | |
| 4 | " | Displacement | 21 | 3.45 | |
| 5 | " | Acceleration | 38 | - | Braking load |
| 6 | " | Displacement | 40 | - | Braking load |
| 7-1 | 2 trucks | Acceleration | 20 | 3.57 | Co-travelling load in about 7 m distance |
| 7-2 | " | " | 17 | 4.17 | |
| 8-1 | 2 trucks | Displacement | 19 | 3.81 | Co-travelling load in about 7 m distance |
| 8-2 | " | " | 19 | 3.75 | |

Tab.3.2 Running Load

| | Large truck 1 | Large truck 2 |
|---|---|---|
| Vehicle weight | 10.46 tf | 10.90 tf |
| Loaded cargo | PHC pile - 600 mm x 9.0 m, 3 pcs | |
| Loaded weight | 10.11 tf | |
| Travelling load | 20.57 tf | 21.01 tf |

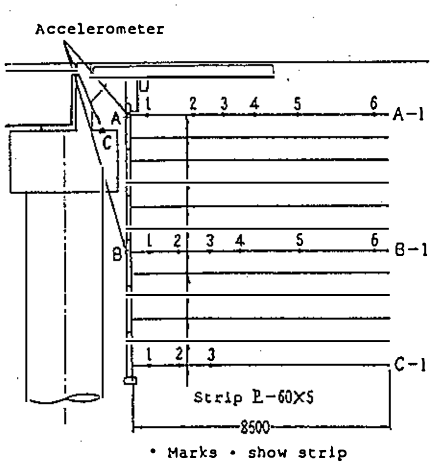
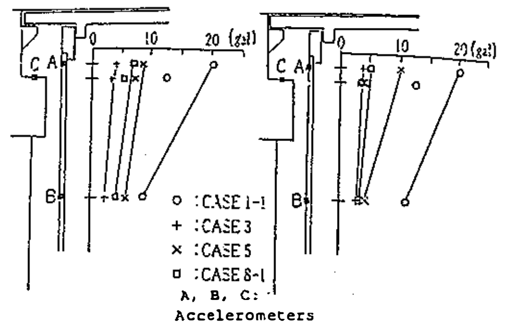


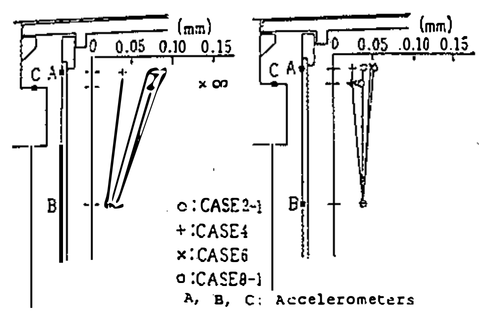

Fig.3.3 Measurement Spots




Distribution of Acceleration

Fig.3.4 Vertical Direction Fig.3.5 Bridge Longitudinal Direction

(2) Displacements on the wall surface, measured by the accelerometers like in the measurement of acceleration, 2-directional components in the bridge longitudinal and vertical directions were measured.
Their distribution state is shown in Fig.3.6 and 3.7.




Distribution of Displacement

Fig.3.6 Vertical Direction Fig.3.7 Bridge Longitudina Direction

## 3.4 Conclusion

Displacements on the wall surface, values measured by accelerometers, were so small as 0.172mm (in the bridge-longitudinal direction) in the center part of the bridge surface in the pier.
We may design such TA abutment taking into consideration two times of theoretical displacement value of the pier to be much safer in order to maintain the necessary clearance in case displacement of pier and TA wall should occur to the opposite direction.