# Partial factors: where to apply them?
## in case of earth retaining wall design practice in Japan

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ABSTRACT: Where to apply partial factors in geotechnical design has been one of the major issues in introducing the limit state design to geotechnical structures. In Eurocode 7, for an example, three different design approaches (namely, DA 1, 2 and 3) have been introduced which have different ways to apply partial factors in design. Several studies on this issue have been conducted in recent years. However, studies comparing the calculated behaviors to the actual behaviors of structures are rear. In this study, actual earth retaining walls that have been observed during the construction are studied. The retaining walls that have been designed using different design approaches are compared with actual observations. Some discussions are made on the pros and cones of the different design approaches for this problem.

## 1 INTRODUCTION

Where to apply partial factors in case of earth retaining wall design? This is the question studied in this paper. This is a controversial issue, because different retaining walls are designed depending on the design approaches (DA) taken. So, several studies on this issue have been conducted in recent years (e.g. Simpson, 2000; Bauduin et al., 2005). However, the study comparing calculated behaviors to actual behaviors of earth retaining walls so far is very few. Therefore, the effect of different DA's on calculated behavior of actual earth retaining wall was studied and the results were compared with actual observations in this paper.

The partial factors are applied in the following three ways in this study:
1) to geotechnical parameters (DA1 in EC7)
2) to actions and calculated total resistance (DA2 in EC7, LRFD)
3) to predicted most likely behavior (design practice in Japan)

## 2 EFFECTS OF PARTIAL FACTORS ON EARTH RETAINING WALL BEHAVIOR

### *2.1 Selected earth retaining walls*

Two actual earth retaining walls were selected for this study. Relatively deep retaining walls are introduced because no observation records are available for shallow ones. However, the deep excavations such as the ones selected here are performed commonly in Japan.

The earth retaining walls are called "Site A" and "Site B" hereafter. The lateral displacements and bending moments of the retaining walls at the final excavation stages will be studied in detail in this paper. The outline of each retaining wall is introduced in this section. The details of the obtained behavior will be described in later sections together with the results of the analyses.

#### *2.1.1 Site A*

Fig. 1 illustrates the cross-section of the earth retaining wall system and ground condition at Site A. The subsoil condition at Site A consists of top soil and Alluvium (Holocene) clay (Ac) with thickness of 8.7 m, Alluvium (Holocene) sand (As) of 3.5m in thickness, Diluvium (Pleistocene) gravel 1 (Dg1) of 8.3m in thickness, and Diluvium (Pleistocene) clay (Dc) with a thickness of 2.9m on Diluvium (Pleistocene) gravel 2 (Dg2) stratum. Geotechnical parameters are also presented in Fig. 1: SPT blow count (SPT N), total unit weight ($\gamma$), cohesion (c) and internal friction angle ($\phi$) are actually measured data, whereas Young's modulus of soil ($E_0$) and coefficient of horizontal subgrade reaction ($k_H$) are estimated by the following equations:

$$E_0 = 2800 \cdot N \quad \text{(cohesionless soil)} \tag{1}$$

$$E_0 = 210 \cdot c \quad \text{(cohesive soil)} \tag{2}$$

$$k_H = \frac{1}{0.3} \cdot \alpha \cdot E_0 \cdot \left( \frac{B_H}{0.3} \right)^{-3/4} \tag{3}$$

where, $\alpha$ is a correction factor for estimation of $E_0$: $\alpha$=1 (cohesionless soil, $E_0$ is estimated by SPT N value) and 4 (cohesive soil, $E_0$ is estimated by c obtained by unconfined compression test) in this example. $B_H$ is equivalent loading width of earth retaining wall (m), and is assumed 10m in this case. The dimensions of the excavation are 16.3m in width and 17.8m in depth, and it was supported by a retaining wall with four levels of struts.

Retaining wall is embedded 6.0m from the final excavation depth. The section of the retaining wall and supports adopted at Site A are described on Table 1 and 2 respectively. The Covering beam (i.e. one at the ground surface) did not restrict movements at the top of the retaining wall, hence it is not considered as a support.

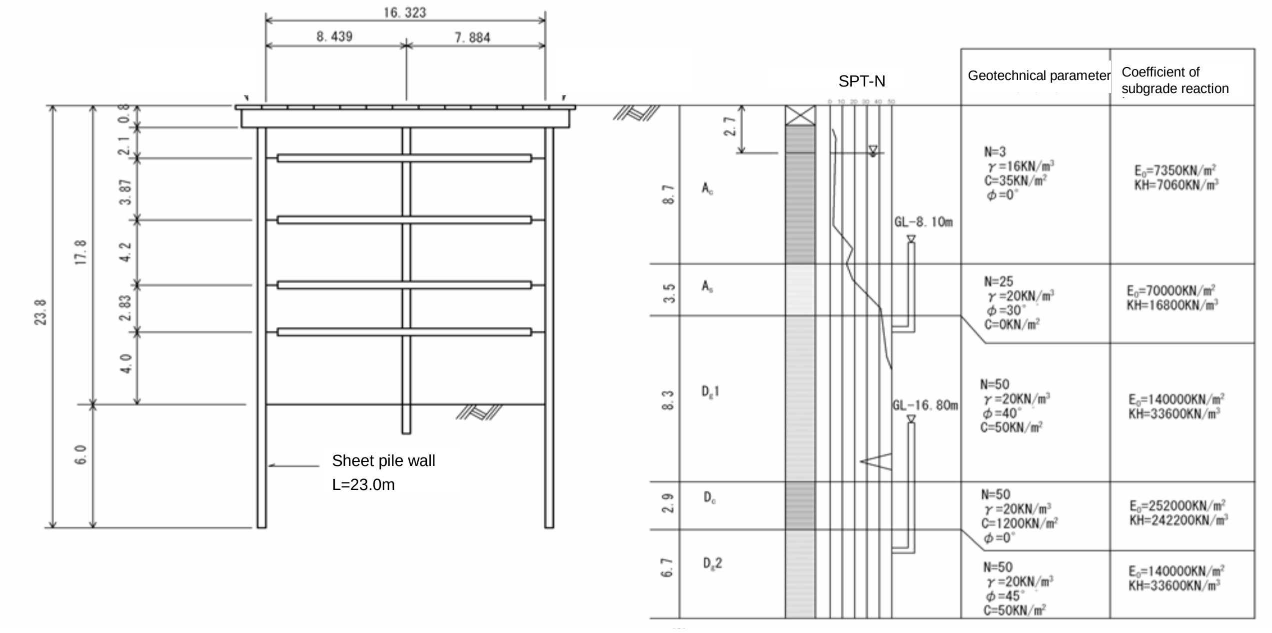



Figure 1 Description of Site A

Table 1 Retaining wall (Site A)

| | Sheet pile wall |
|---|---|
| Specification | FSP-VL |
| I ($m^4$) | $6.3 \times 10^{-4}$ |
| EI ($KN \cdot m^2$) | 79400 |
| Length (m) | 23.0 |

Table 2 Supports (Site A)

| | Supports | Length (m) | Spring constant (KN/m/m) |
|---|---|---|---|
| Covering beam | H-594×302×14×23 | 16.3 | 286000 |
| 1st | H-300×300×10×15 | 15.7 | 160000 |
| 2nd | H-300×300×10×15 | 15.7 | 160000 |
| 3rd | H-300×300×10×15 | 15.7 | 160000 |
| 4th | H-300×300×10×15 | 15.7 | 160000 |

*2.1.2 Site B*

Fig. 2 shows the cross-section of the earth retaining and ground condition at Site B. The subsoil condition at Site B consists of the top soil (sandy) with a thickness of 2.4m, alluvium (Holocene) sand (As) of 9.2m in thickness, alluvium (Holocene) clay 1 (Ac1) of 6.8m in thickness, alluvium (Holocene) clay 2 (Ac2) of 3.2m in thickness, diluvium (Pleistocene) sand (Ds) of 5.5m in thickness, and diluvium (Pleistocene) gravel (Dg) with a thickness of 1.8m on diluvium (Pleistocene) clay (Dc) stratum. Geotechnical parameters are shown in Fig. 2, where, SPT blow count (SPT N), total unit weight ($\gamma$), cohesion (c) and internal friction angle ($\phi$) are actual measurements, whereas Young's modulus of soil ($E_0$) and coefficient of horizontal subgrade reaction ($k_H$) are estimated by Eqs. (1) through (3). The dimensions of the excavation are 9.0m in width and 27.8m in depth. The twelve bracing beams are employed, where the retaining wall is diaphragm wall with 1.2m in thickness. The soil improvement by cement mixing was introduced prior to the excavation at the final excavation bed whose thickness was 2.0m. The geotechnical parameters of this improved soil part are described in the parentheses in Fig. 2. The retaining wall is embedded 5.7m depth from the final excavation depth. The section of the retaining wall and supports adopted at Site B are presented in Tables 3 and 4 respectively. The covering beam at the ground surface did not restrict movements at the top of the retaining wall, hence it is not considered as a support.

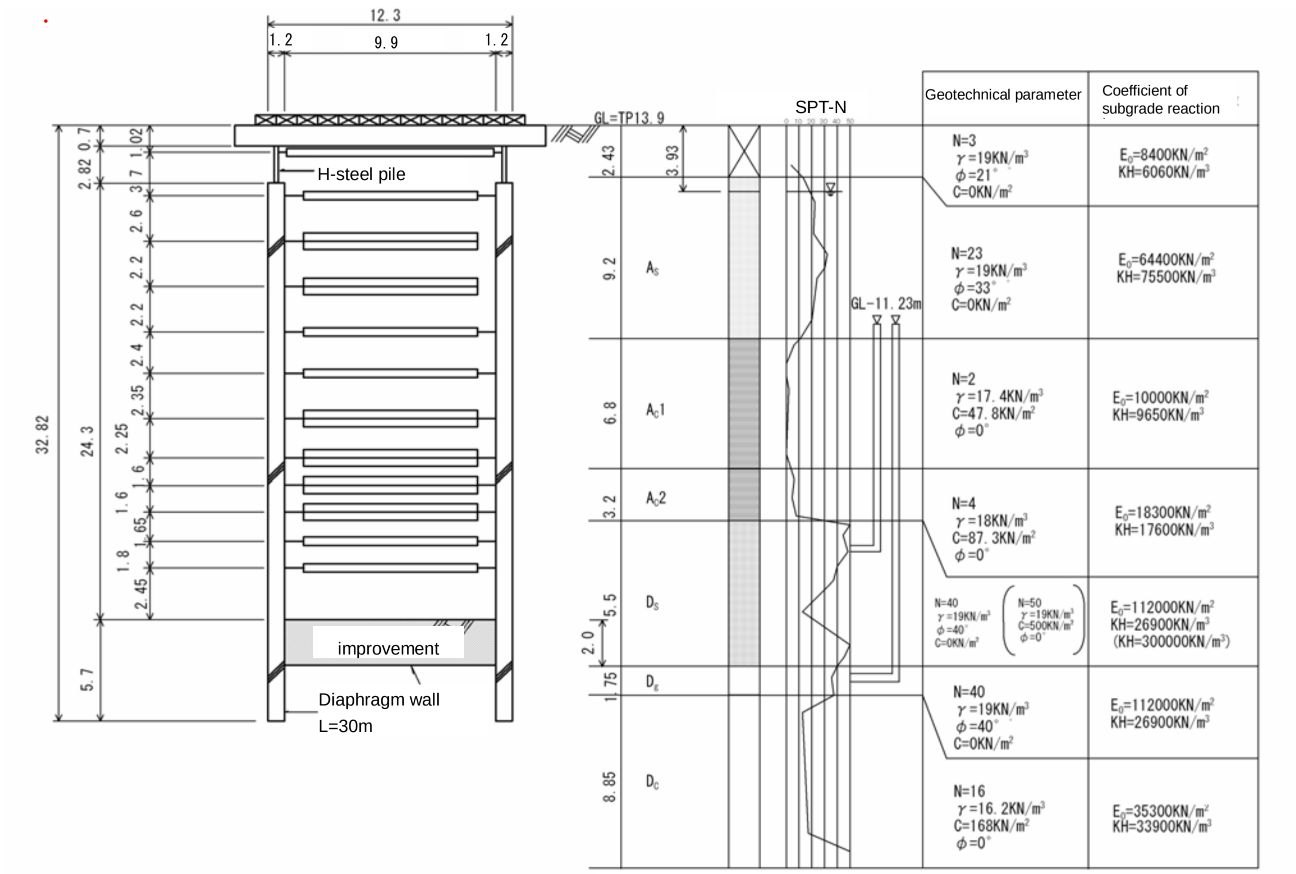



Figure 2 Description of Site B

Table 3 Retaining wall (Site B)

| | H-steel-pile wall | Diaphragm wall |
|---|---|---|
| Specification | H-300×300×10×15 | |
| I ($m^4$) | $2.04\times10^{-4}$ | $1.44\times10^{-1}$ |
| H-steel pile pitch (m) | 1.25 | - |
| EI (KN・$m^2$) | 34300 | 2160000 |
| Length (m) | 2.82 | 30.0 |

Table 4 Supports (Site B)

| | Supports | Length (m) | Spring constant (KN/m/m) |
|---|---|---|---|
| 1 | H-350×350×12×19 | 10.1 | 241000 |
| 2 | H-400×400×13×21 | 9.1 | 338000 |
| 3 | 2H-400×400×13×21 | 9.1 | 675000 |
| 4 | 2H-400×400×13×21 | 9.0 | 677000 |
| 5 | H-400×400×13×21 | 9.0 | 338000 |
| 6 | H-400×400×13×21 | 9.0 | 339000 |
| 7 | 2H-400×400×13×21 | 9.1 | 675000 |
| 8 | 2H-400×400×13×21 | 9.1 | 675000 |
| 9 | 2H-350×350×12×19 | 9.1 | 535000 |
| 10 | 2H-350×350×12×19 | 9.1 | 535000 |
| 11 | H-350×350×12×19 | 9.1 | 268000 |
| 12 | H-350×350×12×19 | 9.2 | 265000 |

## 2.2 *Prediction of the behavior without partial factors (Basic Model)*

The behavior of the earth retaining walls were predicted by a design calculation method using a model of an elastic beam (= retaining wall) on an elasto-plastic foundation, which is used widely for the design of earth retaining walls in Japan (e.g. JGS, 2005). In the calculation, characteristic values of soil parameters are employed (i.e. partial factors = 1.0). It is believed that the characteristic values of soil parameters are almost the average values of the soil parameters.

A comparison of predicted and observed behaviors is shown in Fig.3. The predicted behaviors are reproduced the observations reasonably well; hence this case is adopted as the basic model in this study.

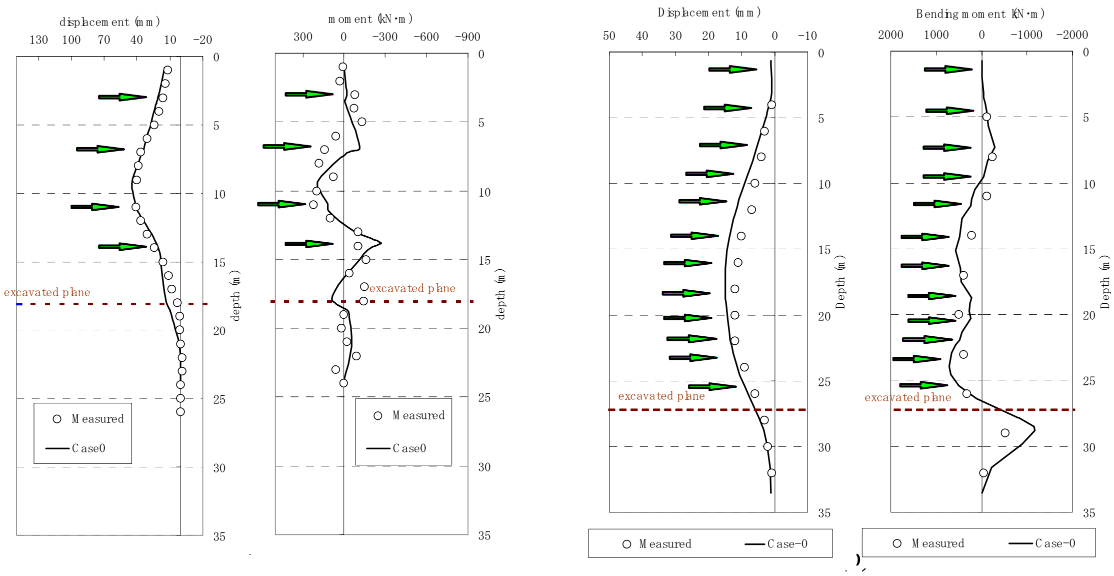



Figure 3 Comparison of predicted behaviors and the observations

## 2.3 *Estimation of the behavior with partial factors*

### *2.3.1 Calculation cases*

Calculated cases are summarized in Table 5, where $\gamma_F$ indicates partial factors for the earth pressures, $\gamma_q$ for overburden, $a_d$ for excavation elevations, and $\gamma_M$ for various soil parameters. Case 0 is the basic model set previously, i.e. all the partial factors are set equal to 1.0. Case 1 is a case that partial factors are applied to only geotechnical parameters. In Case 2, the partial factors are applied to actions as well as to geotechnical parameters, and in Case 3, 4 and 5, only to actions but of different magnitudes (favorable and unfavorable conditions are taken into account).

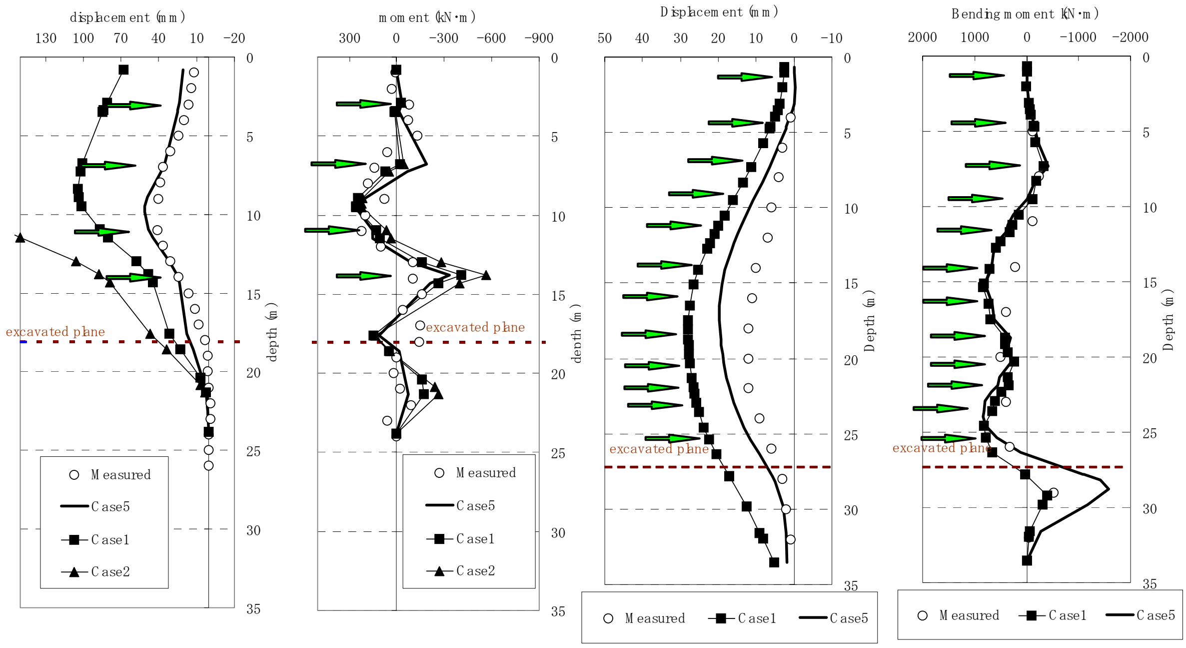
Table 5 Calculation cases

| Partial factors | | Calculation cases 0 | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|---|---|
| $\gamma_F$ | Unfavorable ($K_a$) | 1.0 | 1.0 | 1.1 | 1.3 | 1.35 | 1.3 |
| | Favorable ($K_p$) | 1.0 | 1.0 | 0.9 | 1.3 | 1.4 | 1.5 |
| $\gamma_q$ | (overburden load, 10kN/m$^2$) | 1.0 | 1.0 | 1.5 | 1.0 | 1.0 | 1.0 |
| $a_d$ | Unfavorable (surface) | 0 | 0 | 0 | 0 | 0 | 0 |
| | Favorable (excavation depth) | 0 | 0 | 0 | 0 | 0 | 0 |
| | Favorable (over dig for setting strut) | 1.0 (m) | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| $\gamma_M$ | $\gamma_{\phi'}$ (sandy, $\tan\phi$) | 1.0 | 1.25 | 1.25 | 1.0 | 1.0 | 1.0 |
| | $\gamma_{c'}$ (sandy) | 1.0 | 1.25 | 1.25 | 1.0 | 1.0 | 1.0 |
| | $\gamma_{cu}$ (clayey) | 1.0 | 1.4 | 1.4 | 1.0 | 1.0 | 1.0 |
| | $\gamma_\gamma$ | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |

## *2.3.2 Calculation results*

The calculation results are shown in Fig. 4. The results from Cases 3, 4 and 5 are similar, hence only Case 5 is plotted in Fig.4 as a representative. Furthermore, the comparison of calculated maximum values and observed ones are listed in Table 6. Case 2 at Site B could not be calculated because subgrade reaction exceeded the passive pressure and whole ground becomes plastic state under the excavated plane, i.e. base failure.

It is clearly understood from the calculated results that the behaviors of Case 1 and 2 are far different from the obtained ones in the both Sites A and B. Moreover, the calculated wall displacement as well as the bending moment is far different from the obtained ones in Case 1 at Site B beneath the excavated bed.




(a) Site A (b) Site B

Figure 4 Comparison of calculated behavior with partial factors and the observations

Table 6 Comparison of calculated maximum values with partial factors and the observations

| | Observed behavior | Calculated results | | | | | |
|---|---|---|---|---|---|---|---|
| | | Case 0 | Case 1 | Case 2 | Case 3 | Case 4 | Case 5 |
| Site A | | | | | | | |
| Displacement (mm) | 41 | 45 | 104 | 342 | 59 | 57 | 51 |
| Bending moment (kN･m) | 220 | 268 | 408 | 562 | 351 | 352 | 334 |
| Site B | | | | | | | |
| Displacement (mm) | 12 | 15 | 25 | ∞ | 21 | 22 | 20 |
| Bending moment (kN･m) | 523 | 1,180 | 837 | ∞ | 1,580 | 1,660 | 1,580 |

### *2.3.3 Calculated behavior with partial factors*

#### *(1) Case 1 and Case 2 (partial factors applied to geotechnical parameters)*

Calculated displacements and bending moments are far different from the observed behaviors. The main reason is that the calculated lateral pressures are totally underestimated as a result of the strong non-liner relationship between c and especially $\phi$ of the ground. Estimated lateral pressures and the effect on wall behavior are further reflected below.

*a) Lateral pressures assumed in the design*

The reductions of the geotechnical parameters can affect the value of passive earth pressure as well as active earth pressure significantly. Fig.5 compares the lateral pressure of Case 0 (The basic model), Case 2 and Case 5 after 1st excavation stage at Site A. Not surprisingly, passive earth pressure is underestimated and active earth pressure is overestimated in Case 1 and Case 2.

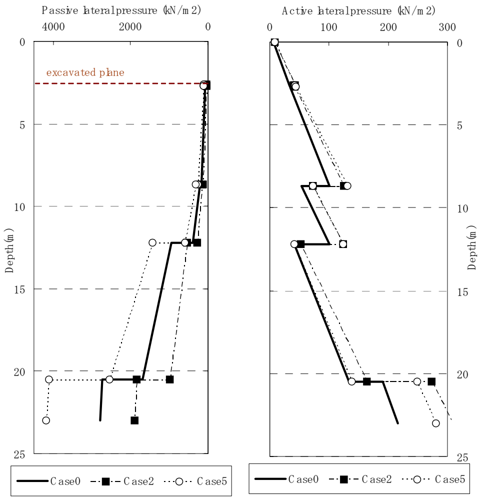



Figure 5 Lateral pressures (1st step, Site A)

*b) Over-estimation of displacement in elasto-plastic analysis*

Fig. 6 describes the mechanism embedded in elasto-plastic analysis used in evaluating the passive subgrade reaction of the retaining wall. Fig. 6 compares the difference between the elastic analysis and the elasto-plastic analysis: In elastic analysis, the mobilized subgrade reaction stress is proportional to the horizontal movement of the ground, whereas in the elasto-plastic analysis, if the subgrade reaction stress exceeds the passive earth pressure, it is redistributed to the lower part of the retaining wall resulting additional displacement. Because of this mechanism, if the passive lateral earth pressure is unrealistically underestimated, the displacement of the retaining wall is overestimated as illustrated in Fig. 7. Hence, the results presented in Table 6 for cases 1 and 2 are to be expected.

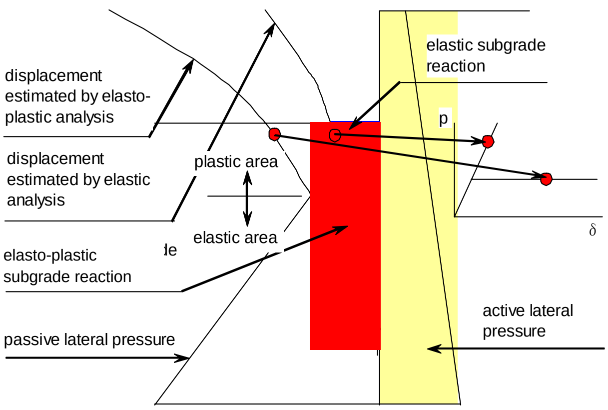



Figure 6 An image of comparison of elastic and elasto-plastic method

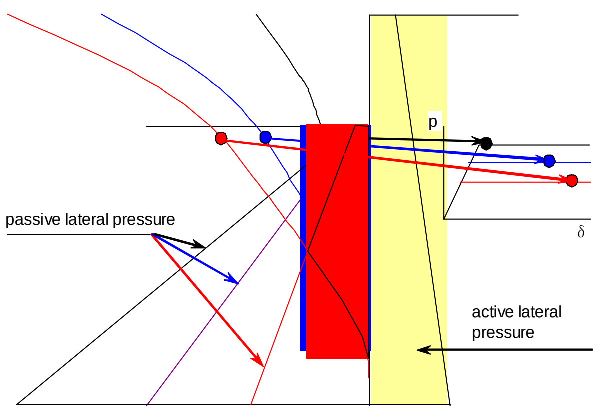



Figure 7 An image of relationship between wall behavior and passive lateral pressure

*(2) Case 3, case 4 and case 5 (partial factors applied to actions and total resistance, favorable and unfavorable conditions are considered)*

Calculated results are simulating the main features of the observations relatively well both in displacements and in bending moments. However, it is speculated that some of the calculations may be unconservative, because larger passive earth pressure may be calculated as shown in Fig. 5. The actual degree of conservatism is unknown because the model uncertainty involved in the design calculation has not quantitatively evaluated. The model uncertainty in the calculation model needed to be quantified by comparing calculated results and observations in many actual case histories.

## 3 DISCUSSION

### *3.1 Where to apply partial factors?*

Pros and cons of the design approaches considered in this study are discussed based on the calculated results presented in the previous sections.

#### *3.1.1 To geotechnical parameters (Cases 1 and 2: material factor approach)*

Because of the strong non-liner relationship between the soil parameters and calculated earth pressures, this approach leads to unrealistically larger/smaller earth pressures. From the view point of supervision and control of the excavation construction, unrealistic calculation results may not provide useful information to designers and contractors, thus this approach is not suitable for excavation retaining wall design.

#### *3.1.2 To total calculated resistance and actions (Cases 3, 4 and 5: LRFD or resistance factor approach)*

Relatively accurate prediction of the actual earth retaining wall behavior was confirmed. However, the actual values of the resistance factors need to be determined based on the accuracy of the method. This uncertainty can only be evaluated from comparing calculated results and the observations in many case histories. Hence, it is considered that predicting the most likely behavior of the structure and introducing safety elements after one see the results might be better than this approach in the present design situation.

#### *3.1.3 To predicted most likely behavior (Case 0: design practice in Japan)*

The deep excavation sites like the ones introduced in this study are quite common in Japan, because Tokyo, Osaka, Nagoya and other metropolitans and big cities are all located on soft ground, and many urban infrastructures need to be placed deep underground. Under such situation, observation, control and supervision of such deep excavations are successfully carried out in Japan by calculating the most likely behavior of the structures. This approach is also fitted for control and supervision during the construction.

### *3.2 Numerical analysis, design and supervision of construction*

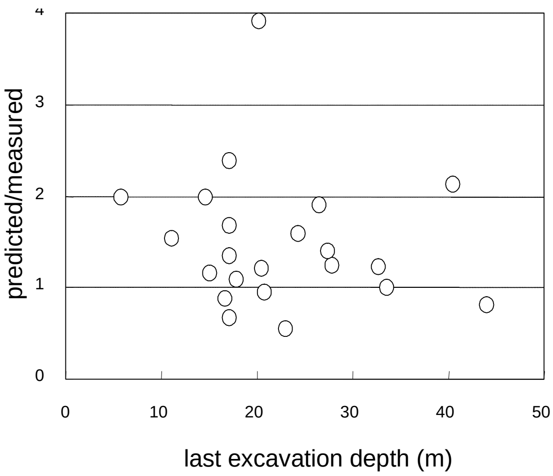
The most likely behavior predicted by numerical analysis with characteristic values (i.e. average values or the most likely values) of soil parameters was proposed as the most favorable DA in this study. However, understanding of the existing uncertainties in the analysis, it should be emphasized that an appropriate supervision understanding these uncertainties is badly needed for the safe excavation construction. In this context, an example of the uncertainty evaluation of the calculation method employed in this study is presented. Furthermore, a philosophy of the excavation supervision in Japan is presented as information for readers.

#### *3.3.1 An example of the uncertainty of the calculation method employed in this study*

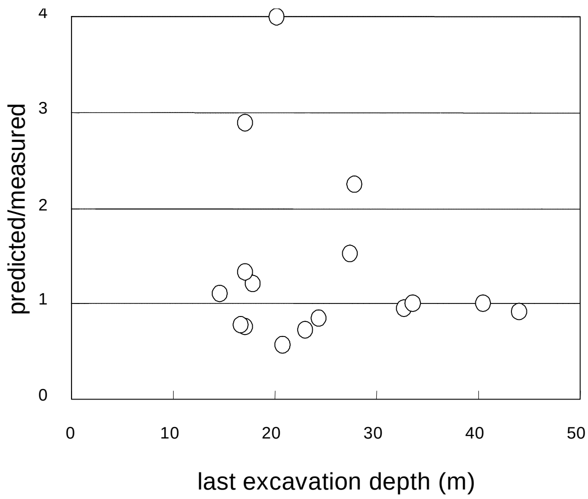
Fig. 8 shows the uncertainty of the calculation method employed in this study. These figures are produced from case-histories of retaining wall construction, which were reported by JGS (1998), JREA (1993) and ACTEC (1994).

The vertical axis shows the ratio of predicted maximum values *versus* the observed ones, thus a value closer to 1.0 implies that the predicted maximum value agrees well with the observed one. On the other hand, the value larger than 1.0 means that predicted behavior overestimates the real one.

According to Fig. 8(a), which shows the comparison of the maximum displacements of 22 retaining walls, the ratios are distributing mostly between 0.5 and 2.0. Furthermore, in the case of Fig. 8(b), which shows the comparison between the maximum bending moments of 16 retaining walls, the ratios distribute mostly between 0.7 and 1.6.



(a) Maximum displacements



(b) Maximum bending moments

Figure 8 An example of the uncertainty of the calculation method employed in this study

*3.3.2 Excavation supervision in Japan*

Fig. 9 shows a concept of excavation construction supervision in Japan. This figure illustrates the relationship between the response behaviors of the retaining wall and progress of an excavation construction. The procedure of the construction control, i.e. the relationship among observations, predictions and actions, are explained as follows:

1) The limit values are the values of the retaining wall and supports when the structure (surrounding structures, the retaining wall and the supports) reach some limit states with some margin of safety.
2) The criteria to take some actions at various stages of the excavation are set based on the predicted behavior of the retaining wall and supports during the design.
3) During the excavation construction, the behavior of retaining wall and supports are supervised based on the criteria set in the previous step.
4) If an observed value exceeds the criterion, the observed values are back analyzed to find the better soil parameter values, and then the future behavior of the excavation is predicted.
5) If the predicted future behavior is smaller than the limit values, the construction is continued with presently planned procedure (e.g. keep the interval of burns as initially panned intervals). However, the criteria are renewed based on the new prediction based on revised soil parameter values.
6) On the other hand, if the predicted behavior exceeds the limit values, a countermeasure should be taken in order to satisfy the behavior not to exceed the limit values. In this case, the criteria should be renewed by the prediction based on the new construction procedure.

This procedure continues until the end of the excavation.

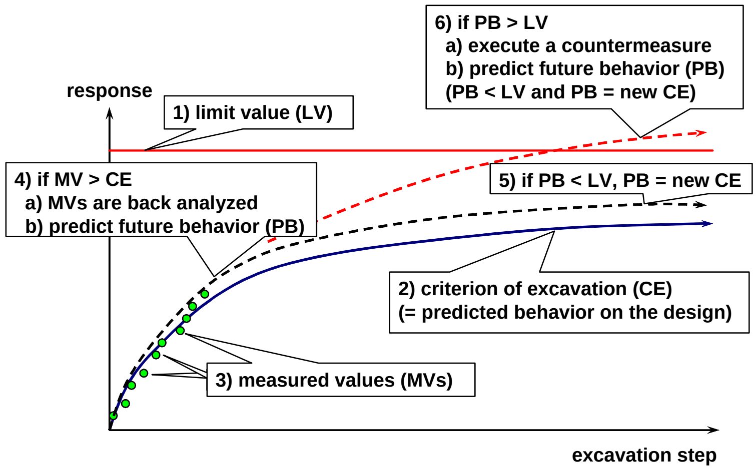



Figure 9 An image of excavation supervision in Japan.

## 4 CONCLUSION

The results of this study are summarized as follows;

- The effect of different DA's on calculated behavior of actual earth retaining wall was studied and the results were compared with actual observations.
- The estimated behaviors using material factor approach (MFA) are far different from the observed ones. From a viewpoint of supervision and control of the excavation construction, unrealistic calculation results (using MFA) may not provide useful information to designers and contractors.
- Relatively accurate prediction of the actual earth retaining wall behavior by load and/or resistance factor approach (LRFA) was confirmed. However, the actual values of the resistance factors need to be determined based on the accuracy of the calculation method.
- Predicting the most likely behavior of the structure using numerical analysis and introducing safety elements after one sees the results, (design practice in Japan), might be better than other DAs in the present design situation.
- In excavation construction with earth retaining walls, the most critical situation would appear during any stages of construction. Thus, an application of observational construction method with predicted most likely behavior by a numerical calculation is the most practical, rational and economical. Many excavation constructions are performed in this way in Japan, and two of the typical examples are presented in this paper.
- The qualitative assessment of the uncertainties of the analysis is inevitable for safety and rational construction control. The evaluated uncertainties of the calculation method employed are also presented in this study.

## ACKNOWLEDGEMENT


The authors are grateful to Kenji Matsui and Masahiro Ishida of Public Works Research Institute for their useful suggestions on the partial factors applications to the earth retaining walls.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