STRUCTURAL STRAIN ESTIMATION OF SEGMENTED SHIELD TUNNEL FOR SEVERE EARTHQUAKES

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SUMMARY

In order to realistically assess the seismic risk of a shield tunnel, the accurate estimate of the structural strains which depends upon the structural details, segment materials, properties of the surrounding soil, the nature of the propagating wave and so on is critical. Emphasis in this study, therefore, has been placed on the analysis of structural strains for segmented shield tunnel under severe earthquake ground motions. The purpose of this study is (1) to define the slippage factor in order to estimate the decrease in tunnel strain resulting from the slippage effect, (2) to furnish the calculation formula for the structural strains of the shield tunnel, and (3) to describe the most effective use of seismic isolation layer for the shield tunnel which is comparable to slippage effect.

INTRODUCTION

Segmented shield tunnels were cracked at the concrete lining wall, but were not damaged at the joints in the axial direction in the 1995 Hyogoken-Nanbu Earthquake. Based on the current design method which was established on the basis of allowable stress criterion, however, all the joints of the shield tunnel in the axial direction could not be designed without the expansion joints for severe ground motion which is given as the level 2 ground motion to be established in Japan since 1995.

This great discrepancy between the non-damage observations of actual joints and the design requirement of expansion joints for stress release result from the stress based design, although the underground structure should be designed and thus based on the structural strains against externally forced ground displacements.

In order to realistically assess the seismic risk of a shield tunnel, the accurate estimate of the structural strains which depends upon the structural details, segment materials, properties of the surrounding soil, the nature of the propagating wave and so on is critical. Emphasis in this study, therefore, has been placed on the analysis of structural strains for segmented shield tunnel under severe earthquake ground motions.

Noting that the shear stress at the interface between the tunnel and the soil makes a large elongation of the tunnel enough to cause the joint failure, the most effective approach is to find some methods how to reduce the shear stress. One approach is to take into consideration the slippage mechanism between the tunnel and the soil, while the other is to introduce the seismic isolation layer between them.

The purpose of this study is (1) to define the slippage factor in order to estimate the decrease in tunnel strain resulting from the slippage effect, (2) to furnish the calculation formula for the structural strains of the shield tunnel, and (3) to describe the most effective use of seismic isolation layer for the shield tunnel which is comparable to slippage effect.

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2. STRUCTURAL MODELS OF SHIELD TUNNEL

2.1 Structural Model

Shield tunnel is composed of many segments which are fasten with bolts. Some segments are made of concrete and the others are of steel elements as shown in Fig.1

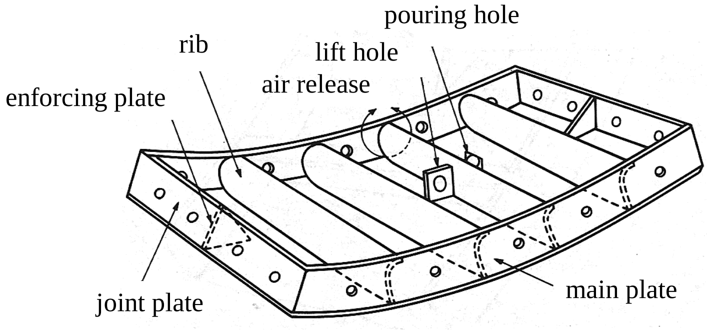
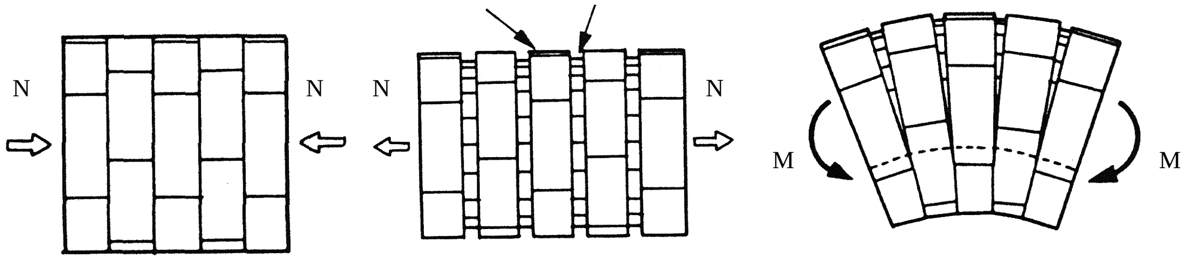


Fig.1 Segment of shield tunnel.

Kawashima[1] describes the characteristics of structural behaviours of shield tunnel, where the tunnel deformations are different in compression mode and tensile mode as shown in Fig.2. This difference also makes the neutral axis deviate to the compression side in bending mode.



(1)compression mode (2)tension mode (3)bending mode

Fig.2 Displacement modes of shield tunnel[1]

Based on this consideration, the cross-sectional area and bending rigidity of shield tunnel are estimated in the equivalent form with Young's modulus. Two different cross-sectional areas are defined in the following way[4] for compression and tension modes, respectively.

$$(EA)_{eq}^{C} = E_s A_s \quad , \quad (EA)_{eq}^{T} = \frac{1}{1 + \dfrac{E_s A_s}{l_s K_J}} E_s A_s \qquad (EI)_{eq} = \frac{\cos^3\phi}{\cos\phi + \left(\dfrac{\pi}{2} + \phi\right)\sin\phi} E_s I_s \qquad with\ \phi = \sin^{-1}\left(\frac{d}{r}\right) \tag{1}$$

in which

- A_s : cross-sectional area of a segment ring
- E_s : Young's modulus of a segment
- l_s : width of a segment
- K_J : spring modulus of one ring joint

  $K_J = k_J \cdot n$

- k_J : spring modulus of a joint
- n : number of joints per one ring joint
- I_s: cross-sectional moment of a segment ring
- r: radius of a tunnel
- d: length from the tunnel center to the cross-sectional center

Consider a seismic wave horizontally travelling and incident to the tunnel with an angle ϕ. The apparent ground strain at the tunnel location is given by

$$\varepsilon_G = \frac{2\pi}{L_a} U_h \chi_v(\phi) \tag{2}$$

where L_a is the apparent wave length along the tunnel axis, while $\chi_v(\phi)$ is the directional parameter given for Rayleigh type surface wave (ν=R) and Love type surface wave (ν=L). Both parameters are given as follows.

$$L_a = \frac{L}{\cos\phi} \quad , \quad \chi_R(\phi) = \cos\phi \quad , \quad \chi_L(\phi) = -\sin\phi \tag{3}$$

The seismic deformation method[4)] provides the structural strains of the tunnel which can be calculated with the apparent ground strain and the conversion factors to be defined hereunder for tension and compression modes, respectively.

$$\varepsilon_{SA} = \alpha_A \varepsilon_G \quad , \quad \varepsilon_{SB} = \alpha_B \varepsilon_G \cdot \left(\frac{2\pi}{L_a} \cdot \frac{D}{2} \right) \tag{4}$$

in which

$$\alpha_A = \frac{1}{1 + \left(\frac{2\pi}{\lambda_A L_a} \right)^2} \quad , \quad \alpha_B = \frac{1}{1 + \left(\frac{2\pi}{\lambda_B L_a} \right)^4} \qquad \text{with} \qquad \lambda_A = \sqrt{\frac{K_A}{(EA)_{eq}}} \ , \ \lambda_B = \sqrt[4]{\frac{K_B}{(EI)_{eq}}} \tag{5}$$

where K_A and K_B are spring modulus for axial and transverse directions.

3.STRUCTURAL STRAIN IN SLIPPAGE

3.1 Shear Stress Acting on the Tunnel Surface

When the earthquake intensity is severe and the free field strain reaches the order of magnitude of 10^{-3} to 10^{-2}, the chance of a slip taking place between the tunnel and the surrounding soil significantly increases causing large strain concentrations at various joints and connections in the shield tunnel system.
The simplified mechanism of slip assumed in this study is that the slip initiate when the shear stress τ exceeds the critical value of $\tau_{cr} = c + \sigma \ tan\phi$ where c= cohesion stress, σ = normal stress at the interface between the tunnel and the soil and ϕ = angle of friction between the tunnel and the soil. Therefore, the critical shear stress increases as the depth of soil cover increases. The possible upper limit of normal stress is controlled by the shadow zone in Fig.3, the normal soil stress of which is effectively acting to the tunnel surface as shown in Fig.4. The above discussion suggests that the critical shear stress behaves not linearly but at least bi-linearly as expressed in Fig.5. If the ground condition installing the shield tunnel can expect a sufficient lateral force, the height of the effective soil pressure zone in Fig.3 is less than several times of the tunnel diameter, so that the possible height of soil cover to the tunnel is approximately 5 ~ 10 meters. According to the Japan Gas Association (JGA), the critical shear stress is recommended to use the value of 0.01N/mm^2 for the gas pipeline which is often installed under the soil cover of 1.2 ~ 1.5 m. Based on these data, the critical shear stress of the shield tunnel installed in several 10 m depth should increase up to 0.05 ~ 0.1 N/mm^2 in proportional to the effective soil depth of 5 ~ 10 m.
The value of 0.05 ~ 0.1 N/mm^2 is assumed for numerical purposes as the critical shear stress in this study.

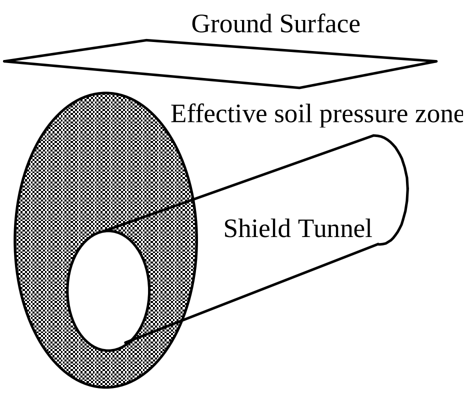


Fig.3 Vertical soil pressure acting on the tunnel.

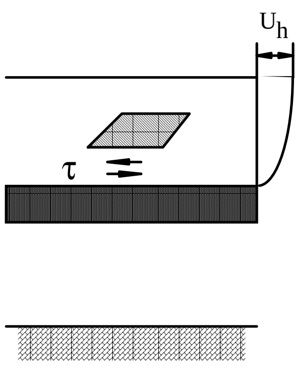


Fig.4 Shear stress acting on the tunnel surface caused by free field motion.

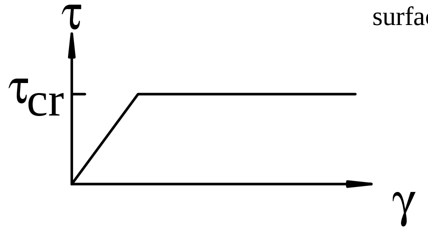


Fig.5 Shear stress and strain relationship of the soil.

3.2 Formulation

The free body diagram of a tunnel segment Δx shown in Fig.6 is subjected to an acceleration $\partial^2 u/\partial t^2$. Applying D'Alembert's principle to the surface, internal and inertia forces acting on this segment, one obtains

$$A\left\{\sigma+\frac{\partial\sigma}{\partial x}\Delta x\right\}+\pi D\tau_G\Delta x=\sigma+\rho A\Delta x\frac{\partial^2 u}{\partial t^2} \qquad (6)$$

Using this formulae, the average shear stress acting on the tunnel surface can be deduced as

$$\tau_G=\frac{(EA)_{eq}}{\pi D}\cdot\frac{2\pi}{L_a}\cdot\alpha_A\cdot\varepsilon_G \qquad (7)$$

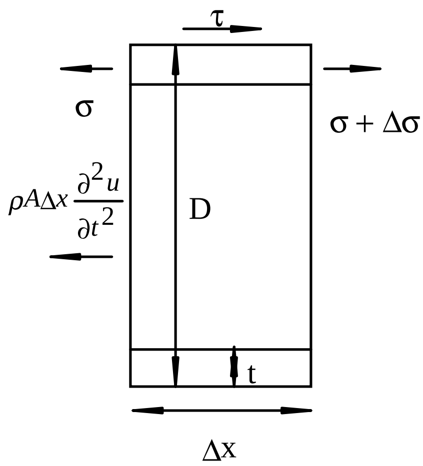


Fig.6 Equilibrium of tunnel element.

Slippage along the interface between the tunnel and the surrounding soil can only take place when the earthquake intensity is large enough that the shear stress produced in the interface reaches the critical value. Noting that τ_G is the maximum shear stress in the soil at the interface, the following criteria can be used to determine whether the slippage will or will not take place at least in some portion along the interface.

If $\tau_G \le \tau_{cr}$, slippage will not take place.

If $\tau_G \ge \tau_{cr}$, slippage will take place.

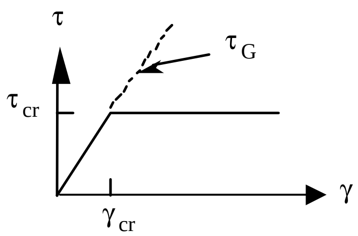


Fig.7 Shear stress-strain relationship of the soil.

This situation can be expressed with the equation for equilibrium in the partial slippage given by

$$\rho A\frac{\partial^2 u}{\partial t^2}-(EA)_{eq}\frac{\partial^2 u}{\partial x^2}=\pi D\tau \qquad (8)$$

where u_S is the tunnel displacement and

$$\pi D\tau=\begin{cases}K_A(u_G-u_s) & \text{;for nonslippage area}\\ K_A u_{cr} & \text{;for slippage area}\end{cases} \qquad (9)$$

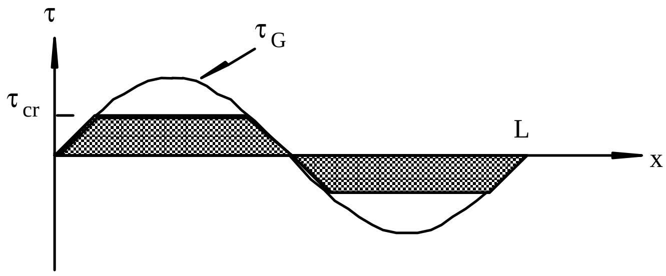


Fig.8 Shear stress distribution along the tunnel axis.

in which u_{cr} is the critical relative displacement initiating the slippage defined by $u_{cr}=\pi D\tau_{cr}/K_A$. Fig.7 shows the schematic profile of the shear strain distribution along the tunnel axis. When the sinusoidal wave form is assumed for the free field motion as $u_G(\eta)=U_h \sin\eta$, the solution of eq.(8) for the non-slippage region in the partial slippage provides the tunnel displacement as

$$u_s(\eta)=U_h\left\{\frac{\kappa^2}{1+\kappa^2}\sinh+\frac{\sinh(\kappa\eta)}{\sinh(\kappa\eta_{cr})}\left(\frac{\sinh(\kappa\eta_{cr})}{1+\kappa^2}-\frac{u_{cr}}{U_h}\right)\right\} \quad \text{with} \quad \kappa=\frac{L_a}{2\pi}\lambda_A \qquad (10)$$

and that of eq.(8) for the slippage region furnishes the tunnel displacement as

$$u_S(\eta)=\kappa^2 u_{cr}\left(-\frac{\eta^2}{2}+\frac{\eta\pi}{2}\right)+C \qquad (11)$$

Eliminating C at the location of $\eta=\eta_{cr}$, one can obtain the critical relative displacement ratio S expressed with a parameter η_{cr} to be identical to the location of slippage initiation.

$$S=\frac{u_{cr}}{U_h}=\frac{1}{1+\kappa^2}\cdot\frac{\kappa\cos\eta_{cr}\tanh(\kappa\eta_{cr})+\sin\eta_{cr}}{\kappa\left(\frac{\pi}{2}-\eta_{cr}\right)\tanh(\kappa\eta_{cr})+1} \qquad (12)$$

The maximum tunnel strain when the partial slippage is taking place can be calculated by

$$\varepsilon_s = \left|\frac{\partial u_s}{\partial x}\right|_{\eta=0} = \frac{2\pi}{L_a}\cos\phi\left|\frac{\partial u_s}{\partial \eta}\right|_{\eta=0} \tag{13}$$

Substituting eqs.(10) and (12) into eq.(13), the maximum tunnel strain in the partial slippage is expressed by

$$\varepsilon_S = q_s \alpha_A \varepsilon_G \quad \text{with} \quad q_s = 1 - \frac{S\left(1+\kappa^2\right) - \sin\eta_{cr}}{\kappa \sinh(\kappa\eta_{cr})} \tag{14}$$

and the relative displacement between the soil and the tunnel motion is provided by

$$\Delta = \left(1 - q_s^* \alpha_A\right) u_G \quad \text{with} \quad q_s^* = \left(1+\frac{1}{\kappa^2}\right)\left[\sin\eta_{cr} - S\left\{1 - \left[\frac{\pi^2}{8} + \left(\frac{\eta_{cr}^2}{2} - \frac{\pi}{2}\eta_{cr}\right)\right]\kappa^2\right\}\right] \tag{15}$$

in which

$$\frac{\tau_G}{\tau_{cr}} = \frac{1-\alpha_A q_s^*}{S} \tag{16}$$

The slippage factors, q_S and q^*_S, are dependent on the conversion factor a_A. Those results suggest that the pipe strain distribution in slippage deviates from the sinusoidal wave form. The simplified approximate formula for q_S and q^*_S is also developed by the author who assumed the shape of structural strain distribution under slippage to be sinusoidal in the following way.

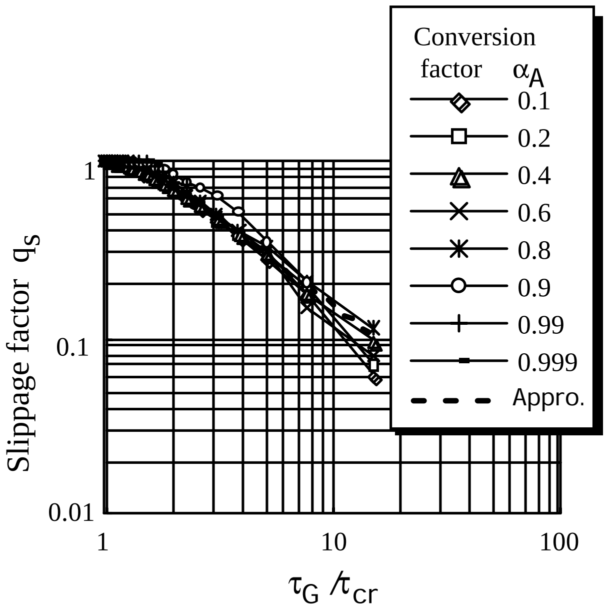
$$\tau_G \geq \tau_{cr}\ ,\quad q = 1 - \cos\xi + \left(\frac{\pi}{2} - \xi\right)\sin\xi\ , \qquad \xi = \arcsin\left(\frac{\tau_{cr}}{\tau_G}\right) \quad ; \quad \tau_G \leq \tau_{cr} \quad q = 1 \tag{17}$$

$$\tau_G \geq \tau_{cr}\ ,\quad q^* = \left(1 + \frac{\pi^2}{8} - \frac{\xi^2}{2}\right)\cdot\sin\xi - \xi\cdot\cos\xi \quad ; \quad \tau_G \leq \tau_{cr} \quad q^* = 1 \tag{18}$$

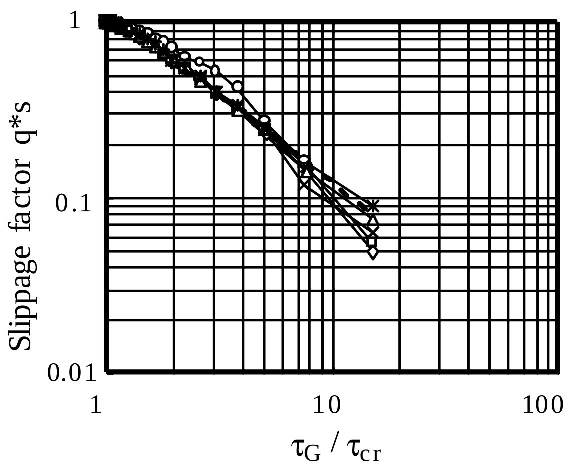
Fig.9 shows the slippage factors for the shear stress normalized by the critical shear stress, in which the simplified approximate formula (broken line) expresses an average value around the numerical results of rigorous analyses. These trends are applicable for both slippage factors. Once the tunnel strains are obtained, various forces acting on a tunnel segment are given in Table 1.

Table 1 Formula to obtain forces from tunnel strains.

Forces	Formula
Tensile force	$P^T = (EA)_{eq}^T \varepsilon_{SA}^T$
Comprssive force	$P^C = (EA)_{eq}^C \varepsilon_{SA}^C$
Bending moment	$M = \left(\frac{2}{D}\right)(EI)_{eq}\,\varepsilon_{SB}$
Shear force	$Q = \frac{2\pi}{L_a}\frac{2}{D}(EI)_{eq}\varepsilon_{SB}$



(1)Slippage factor for tunnel strain



(2)Slippage factor for relative displacement

Fig.9 Slippage factors

4. SEISMIC ISOLATION LAYER FOR SHIELD TUNNEL

4.1 Formulation

When a seismic isolation layer is installed between the tunnel and the surrounding soil, the ground displacement has two components of shear deformation which are contributed from surrounding soil and seismic isolation layer shown in Fig.10. The combined soil stiffness K_A^* is evaluated as the series system of the stiffness K_A of surrounding soil and the stiffness K_B of seismic isolation layer in the following way.

$$K_A^* = \frac{K_A K_B}{K_A + K_B} \tag{19}$$

Since the equation for equilibrium discussed in Section 3.2 is valid for the tunnel element in this section, the following equation is applied for the tunnel element having a seismic isolation layer.

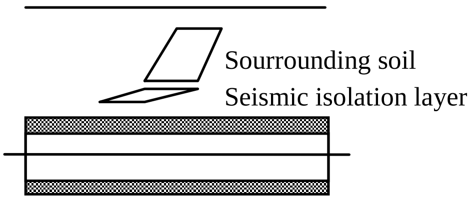


Fig.10 Ground displacement combined with surrounding soil and seismic isolation layer.

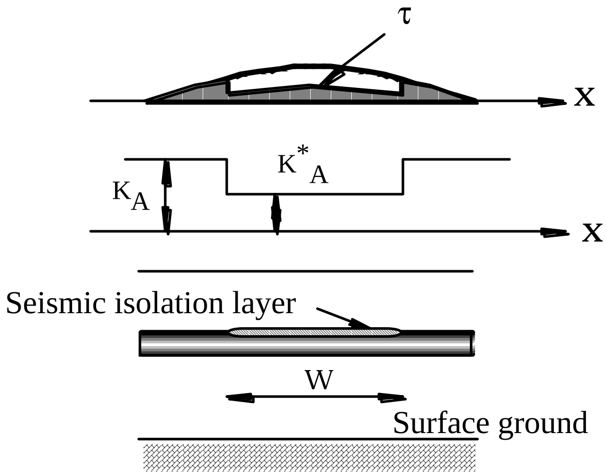


Fig.11 Structural model of shield tunnel and seismic isolation layer.

$$\rho A \frac{\partial^2 u_s}{\partial t^2} - (EA)_{eq} \frac{\partial^2 u_s}{\partial x^2} = \pi D \tau \tag{20}$$

$$\pi D\tau = \begin{cases} K_A(u_G - u_S) & ; \ 0 \le x \le \frac{L_a}{4} - \frac{W}{2} \\ K_A^*(u_G - u_S) & ; \ \frac{L_a}{4} - \frac{W}{2} \le x \le \frac{L_a}{2} \end{cases} \tag{21}$$

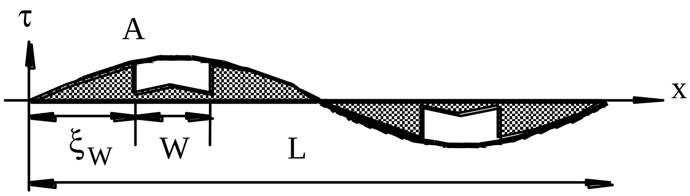


Fig.12 Shear stress distribution.

in which W is the length of a seismic isolation layer. Fig.11 shows the schematic profile of the tunnel and seismic isolation layer installed at the interface between the tunnel and the surrounding soil.
Solving the eq.(20), one obtains the following general solutions.

$$u_S(\xi) = \alpha_A u_G \sin\xi + S\sinh\left(\frac{\lambda_A \xi}{k_L \cos\phi}\right) + T\cosh\left(\frac{\lambda_A \xi}{k_L \cos\phi}\right) \qquad u_S^*(\xi) = \alpha_A^* u_G \sin\xi + S^*\sinh\left(\frac{\lambda_A^* \xi}{k_L \cos\phi}\right) + T^*\cosh\left(\frac{\lambda_A^* \xi}{k_L \cos\phi}\right) \tag{22}$$

in which ϕ is incident angle to the tunnel, and

$$\alpha_A = 1 \Big/ \left\{1 + \left(\frac{k_L \cos\phi}{\lambda_A}\right)^2\right\}, \ \alpha_A^* = 1 \Big/ \left\{1 + \left(\frac{k_L \cos\phi}{\lambda_A^*}\right)^2\right\}, \ \lambda_A = \sqrt{\frac{K_A}{(EA)_{eq}}}, \ \lambda_A^* = \sqrt{\frac{K_A^*}{(EA)_{eq}}}, \ k_L = \frac{2\pi}{L} \tag{23}$$

The boundary condition requires the continuity of displacement and bending angle at the point A in Fig.12, while the seismic wave given by a sinusoidal wave form must satisfy the wave conditions at $x=0$ and $x=\pi/2$.

$$u_s(\xi_W) = u_s^*(\xi_W), \ \left[\frac{\partial u_s}{\partial \xi}\right]_{\xi=\xi_W} = \left[\frac{\partial u_s^*}{\partial \xi}\right]_{\xi=\xi_W} \quad at \quad \xi_W = \frac{L_a}{4} - \frac{W}{2} \quad and \quad u_s(0) = 0, \ \left[\frac{\partial u_s^*}{\partial \xi}\right]_{\xi=\frac{\pi}{2}} = 0 \tag{24}$$

Then the maximum axial strain of the shield tunnel can be obtained as the summation of ground shaking part and strain reduction part due to seismic isolation layer in the following way.

$$\varepsilon_m = \alpha_A\ \varepsilon_G + \lambda_A\ S \tag{25}$$

in which the parameters S and S^* can be evaluated from the following equations.

$$\begin{bmatrix} \sinh\zeta\ , & -\sinh\zeta^* + \dfrac{\cosh\zeta^*}{\tanh\left(\zeta^*\dfrac{\pi}{2\xi_W}\right)} \\ \dfrac{\zeta}{\xi_W}\cosh\zeta\ , & -\dfrac{\zeta^*}{\xi_W}\left(\cosh\zeta^* - \dfrac{\sinh\zeta^*}{\tanh\left(\zeta^*\dfrac{\pi}{2\xi_W}\right)}\right) \end{bmatrix} \begin{bmatrix} S \\ S^* \end{bmatrix} = \begin{bmatrix} \left(\alpha_A^* - \alpha_A\right) u_G \sin\xi_W \\ \left(\alpha_A^* - \alpha_A\right) u_G \cos\xi_W \end{bmatrix} \tag{26}$$

where

$$\zeta = \frac{\lambda_A}{k_L \cos\phi}\xi_W\ ,\ \ \zeta^* = \frac{\lambda_A^*}{k_L \cos\phi}\xi_W\ ,\ \xi_W = k_L\left(\frac{L}{4} - \frac{W}{2}\right)\cos\phi - \omega\ t \tag{27}$$

4.2Numerical Study

The shield tunnel is composed of steel segments with 2.75m in diameter and 12.8 cm in thickness (which is equal to the equivalent cross-sectional stiffness $(EA)_{eq}=3.36x10^7KN)$. The values of 0.01, 0.05 and 0.1 N/mm^2 is used as the parameter for critical shear stress in slippage, while the values , ζ_W, of 0.1 and 0.2 is adopted as the ratio of the width of seismic isolation layer per seismic wave length. The values, ζ_G, of 0.001, 0.01 and 0.1 are also selected as the ratio of reduced soil stiffness of seismic isolation layer per original soil stiffness. Basic parameters of seismic wave length, soil stiffness and response spectrum at baserock are assumed only for the numerical purposes in this study to be identical to those of the guideline of seismic design method of Japan Water Works Association (JWWA)[5)].

For instance, the level 2 ground motion in JWWA is estimated with the response spectrum of seismic velocity at the baserock as shown in Fig.13.

Fig.14 (1), (2) and (3) show the relationship between the structural strains and typical period of the ground for various stiffness and width of seismic isolation layer and for various critical shear stresses in slippage. The symbols, ε_G, ε_{ns}, ε_s, ε_m, are apparent ground strain given by eq.(2), structural strains given by eq.(14) in non-slippage ($q=1$) and in slippage, and axial strain in seismic isolation layer given by eq.(25), respectively.

The following results can be summarised.

Fig.14 (1) shows that the extremely degraded soil stiffness used for seismic isolation layer is not always effective for the reduction of shear stress acting between the tunnel and the surrounding soil.

Fig.14 (2) shows that for the surrounding soil of the critical shear stress less than $0.1N/mm^2$, slippage effect is more remarkable in longer typical period than the reduction effect due to seismic isolation layer; and

Fig.14(3) shows that the reduction effect due to seismic isolation layer is comparatively effective in shorter typical period for the width of layer longer than 20% of the seismic wave length.

These results suggest that the seismic isolation layer method is effective for the ground with comparatively short period, while the method taking the slippage effect is more activated in the longer period of the ground.

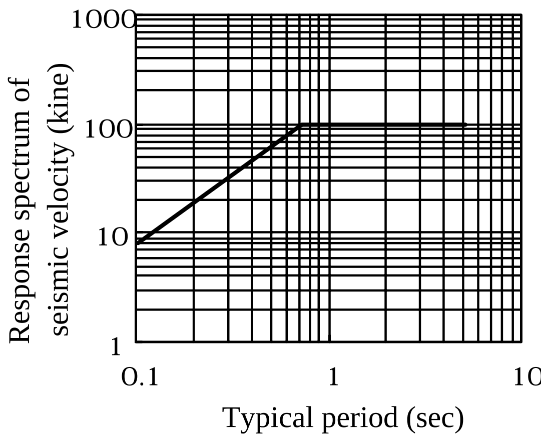
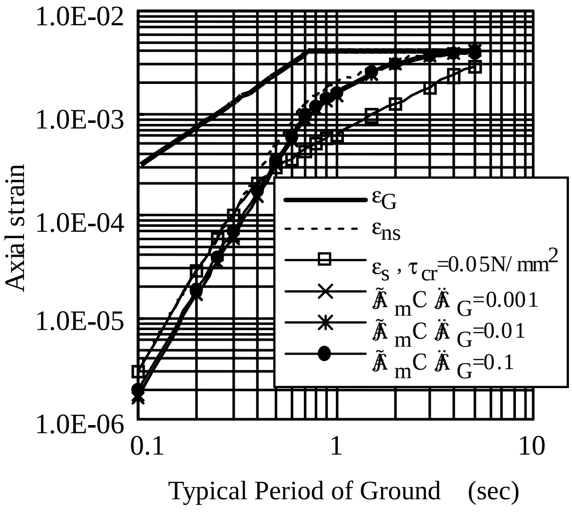
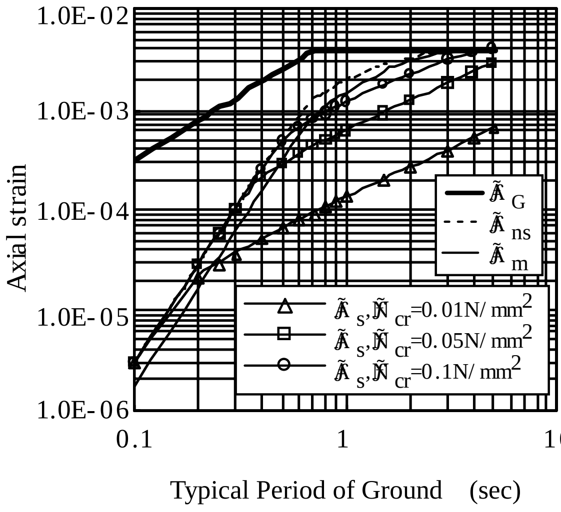


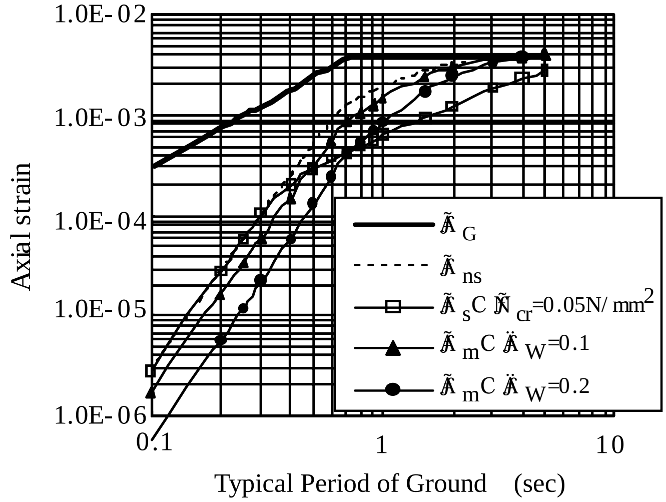
Fig.13 Response spectrum of seismic velocity given by JWWA.



(1)Various soil stiffness



(2)Various critical shear stress



(3)Various width

Fig.14 Comparison between slippage effect and stress reduction effect of seismic isolation layer.

5.CONCLUSIONS

In order to estimate the axial strains of shield tunnel, both slippage and seismic isolation mechanism are taken into consideration for severe earthquake (which is called Level 2 ground motion in Japan). Based on he simplified design formula developed herein, several numerical results are furnished and summarised as follows.

1) for the surrounding soil of the critical shear stress less than 0.1N/mm^2, slippage effect is more remarkable in longer typical period than the reduction effect due to seismic isolation layer;

2)the reduction effect due to seismic isolation layer is comparatively effective in shorter typical period for the width of layer longer than 20% of the seismic wave length; and

3)the extremely degraded soil stiffness used for seismic isolation layer is not always effective for the reduction of shear stress acting between the tunnel and the surrounding soil.

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