A Study on Vertical Distribution of Shear Force Coefficient for Seismic Design of Seismically Isolated Buildings

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SUMMARY:
This paper proposes an evaluation method of shear force coefficient for seismic design of seismically isolated buildings. This evaluation method is based on earthquake response analyses on seismically isolated buildings and is obtained from the relationship between vertical distribution of shear force coefficient and response amplification caused by higher-mode responses. In addition, the proposed evaluation method of response amplification is valid for evaluation of overturning moment. Furthermore, this evaluation method is applied to various restoring force model of isolation layer such as Tri-linear or Ramberg-Osgood characteristics by using approximation method. In conclusion, this proposed method evaluates vertical distribution of shear-force coefficient for various kinds of seismically isolated buildings and shows suitable correspondence compared with the results of non-linearity time history analysis.

Keywords: Seismic isolation, Shear force coefficient, Higher modal response, Isolation ratio, Non-linearity factor

1. INTRODUCTION
Since the Hyogoken-Nanbu Earthquake (1995), in Japan, seismic isolation system has been esteemed for structural safety and maintaining of functional capacities. Furthermore, the effects of response reducing of Seismically Isolated (SI-) buildings have been demonstrated in Tohoku-Chiho Taiheiyo-Oki Earthquake (2011), thereby it will possible to spread more and more in the future. Generally, it is ideal for SI-buildings that the superstructure behaves as rigid-body. Therefore, in several countries, there are seismic design codes (UBC, FEMA etc...) using Equivalent Linearizing Analysis Method without Non-linear Time History Analysis (NTHA) for SI-buildings which are modeled as an equivalent single degree of freedom system. Seismic force at i-th mass \( F_i \) are calculated as inverted triangle distribution which evaluated by the weight and height distribution of the buildings as follows:

\[
F_i = Q_{iso} \cdot w_i \cdot h_i / \sum (w_i \cdot h_i)
\]  

where \( Q_{iso} \) is the base shear of isolation layer, \( w_i \) is the weight at level \( i \), \( h_i \) is the distribution of at that level \( i \). On the other hand, in Japan, the shear force coefficient on the superstructure in the recommendation for design of SI-buildings (AIJ 2001) is as follows:

\[
\alpha_i = \alpha_f + \alpha_l \cdot \bar{a} \cdot \alpha_s
\]

where \( \alpha_s \), \( \alpha_f \) and \( \alpha_l \) are the shear force coefficient of \( i \)-th story of SI-buildings, elastomeric isolator and elasto-plastic dampers, respectively. \( \alpha_f \) is the optimum yield shear force coefficient distribution considered the natural periods and weight distribution. \( \alpha_s \) is given by Eq.1.3. Where \( N \) is the number of structure story, \( \bar{a} \) is given by Eq.1.4.
\[ a_i = \left\{ \frac{\alpha - 1}{N - 1} \right\}^* + \frac{N - \alpha}{N - 1} \]  

\[ \alpha = 3.1238 - 0.1238b_i \quad 1 \leq b_i < 10 \]
\[ \alpha = 2.0127 - 0.0127b_i \quad 10 \leq b_i < 80 \]
\[ \alpha = 1.0 \quad 80 \leq b_i \]  

where \( b_i \) is the ratio of horizontal stiffness of the first story of superstructure in base fixed condition to that of dampers in isolated condition (\( = k/k_s \)). However, Japanese design code for SI-buildings (MVIT 2001) is as follows:

\[ \alpha = \gamma \cdot \frac{Q_{\text{iso}}}{M \cdot g} \cdot \frac{A_i (Q_h + Q_v) + Q_v}{Q_h + Q_v + Q_e} \]  

where \( M \) is the total mass of the structure, \( g \) is the gravitational acceleration, \( \gamma \) is the multiplier including the effects of aging, temperature and property dispersion by manufacturing, \( Q_h \), \( Q_v \) and \( Q_e \) is shear force of elastomeric isolators, elasto-plastic dampers and fluid dampers, respectively. \( A_i \) is the optimum yield shear force coefficient distribution. It is a characteristic point compared with the standard of other countries, this standard is enable to design SI-buildings up to 60 meters in height. However, it has been pointed out that the above-mentioned method likely underestimates the seismic response (M. Takayama et al. 2002). The following cases are more possibility; 1) Superstructure doesn't behave as rigid-body. 2) Seismic isolator has high-stiffness, it has been confirmed that higher-mode responses is generated. In other words, seismic forces and floor accelerations of the superstructure are amplified more than ideal isolation system. This phenomenon counteracts the purpose of isolation system. Thus, this paper proposes a response amplification factor \( \beta \) considering higher-mode responses to evaluate the seismic response properly.

2. VERTICAL DISTRIBUTION OF LATERAL SHEAR FORCE COEFFICIENT

This section shows an evaluation method for the response amplification of shear force coefficient distribution of SI-building. The degree of amplification is calculated from a large number of Non-linearity Time History Analyses (NTHA) and is formulated by measure which evaluates the structural characteristics. The proposed method shows suitable correspondence compared with the results of NTHA.

2.1. Non-linear Time History Analysis

A large number of NTHA is carried out on 10-story shear-type model with Bi-linear restoring force model as seismic isolation layer. Firstly, superstructure is decided as base fixed model, the mass at each story is set to 1,000 tons, story stiffness distribution is decided as 1/2 ratio trapezoid. The first natural period \( T_0 \) is set to 0.2-1.6 sec. Resisting force characteristics of superstructure is linear. The initial structural damping is applied using stiffness-proportional damping and the first mode damping ratio is set to 0.02. Then, as isolated model, the position of isolation layer is set to first story. Isolation layer which is composed of elastomeric isolator and elasto-plastic hysteretic dampers. Lateral stiffness of elastomeric isolator \( k_i \) is calculated from the isolation period \( T_i \) which is set to 4.0 sec. Lateral stiffness of hysteretic damper is \( k_s = Q_s / \delta_s \), \( Q_s \) as the yield strength of hysteretic damper is given by shear coefficient of damper \( \alpha_s \) which is set to 0.01-0.10 by each 0.01. \( \delta_s \) as the yield displacement of hysteretic damper is 0.1, 1.0 and 3.0 cm. These models and parameters are shown in Table.2.1 and Fig.2.1. Then input earthquakes are artificial earthquake ground motions for design which considered site amplification factor \( G_s \) corresponds to 1.23(const.), 1st class and 2nd class in Japanese seismic code. Fig.2.2 shows acceleration response spectrum of artificial earthquake ground motions when \( G_s \) is assumed 2nd class. The analysis results are shown in Fig.2.3. As the amount of dampers \( \alpha_s \) and the
natural periods of the superstructure are increasing, the distribution of shear coefficient is amplified. Especially, it is remarkable in the middle floors over the 4-th floor.

Table 2.1. Structure model and analysis parameters.

<table>
<thead>
<tr>
<th>Structure model</th>
<th>Analysis parameters (5,940 patterns)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Analysis model</td>
<td>10-story shear model</td>
</tr>
<tr>
<td>Mass distribution</td>
<td>Each 1,000 [ton]</td>
</tr>
<tr>
<td>Stiffness distribution</td>
<td>1/2 ratio trapezoid.</td>
</tr>
<tr>
<td>Restoring characteristics</td>
<td>Elastic (h=2%)</td>
</tr>
<tr>
<td>Restoring characteristics of Isolation devises</td>
<td>Isolator: elastic</td>
</tr>
<tr>
<td>Periods of Isolation</td>
<td>4.0 [sec]</td>
</tr>
</tbody>
</table>

Analysis parameters:

- Yield displacement of hysteresis damper \( \delta \): 0.1, 1.0, 3.0 [cm]
- Natural periods of superstructure base-fixed condition \( T_0 \): 0.2, 0.24, 0.28, 0.32, 0.36, 0.4, 0.5, 0.6, 0.8, 1.2, 1.6 [sec]
- Artificial earthquake ground motions for design
- Random pulse \( \times 3 \), Elcentro (1940) NS phase, Hahinohe (1968) EW phase, Kobe (1995) NS phase

2.2. Amplification Factor \( \beta \)

In the cases of using hysteresis damper, shear coefficient distribution is represented by the following equation which is adopted by the form of Eq.1.5:

\[
\alpha_i = \alpha_f + \beta_i \cdot A_i \cdot \alpha_s
\]  

(2.1)

where \( \beta_i \) is a amplification factor for \( A_i \) distribution which gives a correct distribution of response amplification by hysteresis damper. If \( \beta_i \) equals 1, Eq.2.1 is identical to Eq.1.5 (It don’t use fluid damper; \( Q_v=0 \)). If \( \beta_i \) is given by Eqs.1.3-1.4, Eq.2.1 is identical to Eq.1.2. Fig.2.4 shows the composition of Eq.2.1. If the distribution of shear force coefficient is calculated by Eq.2.1, response amplification of the shear force can be perceived by evaluating of the amplification factor \( \beta_i \). Thereby Eq.2.2 is replaced by Eq.2.1 and substitutes the results of NTHA to analyze the distribution of \( \beta_i \).

\[
\beta_i = \frac{\alpha_i - \alpha_f}{A_i \cdot \alpha_s}
\]  

(2.2)
Fig. 2.5 shows the distribution of $\beta_i$. In these cases, $\beta_i$ is greater than 1. In other words, evaluation method by using $A_i$ distribution only is insufficient in association with the representation of response amplification for the height direction. Here, approximation method of $\beta_i$ into a linear line is as follows:

$$
\beta_i = \frac{1}{N} \left[ \frac{1}{N} \sum_{k=1}^{N} \frac{(\beta_i - \overline{\beta})}{(N-1)} \right] i + \frac{(N-1)}{(N-1)}
$$

(2.3)

where $\overline{\beta}$ is the value of $\beta_i$ on the top story which is obtained from a linear approximation of $\beta_i$ distribution shown in Fig. 2.5 by the method of least squares. But $\beta_i$ is set that the first layer of the approximation line ($\beta_1$) equals 1. This approximation method is shown in Fig. 2.6.

2.3. Isolation ratio $I$ and Non-linearity factor $NL$

$\beta_i$ is calculated from the results of NTHA and is formulized by Isolation-ratio $I$ and Non-Linearity factor $NL$ which are proposed by Skinner (skinner et al. 1993). $I$ is the ratio between the natural period of the superstructure in the fixed-base condition ($T_0$) to that of seismic isolation layer corresponding to the initial stiffness ($T_{b1}$). The Isolation ratio $I$, which governs so many aspects of seismic response, is a measure of period shift produced by isolation.

$$
I = \frac{T_{b1}}{T_0}
$$

(2.2)

$NL$ defines Non-linearity of Isolation layer as quantitative. $NL$ is the ratio of the maximum loop offset, from the secant line joining the points, to the maximum offset of axis-parallel rectangle through these points, i.e. $P_1/P_2$. Hence $NL$ increases from 0 to 1 as the loop changes from zero-area shapes to a rectangular shape. For a Bi-linear isolator, this is equivalent to the ratio of loop area $A_b$ to that of the rectangle. Definition of $NL$ is given as follows:

$$
NL = \frac{R}{P_2} = \frac{A_b}{4 \cdot \delta_b \cdot Q_b} = \frac{\pi \cdot h_{eq}}{2}
$$

(2.3)

$NL$ is proportional to the hysteretic damping factor $h_{eq}$ for Bi-linear hysteretic loops. Fig. 2.7 shows hysteresis loop defines Non-linearity factor $NL$. In this paper, consideration of generality, $NL$ replaces with the equivalent viscous damping ratio $h_{eq}$.

The relationship between isolation ratio $I$ and $\beta$ is shown in Fig. 2.8 which is classified by each equivalent viscous damping ratio $h_a$. Diamond-shaped mark is moving average $\overline{\beta_{MA}}$ which is average...
of \( \beta \) in the range of Isolation ratio \( I \) from 0 to 0.8, then moved by each 0.4. \( \beta \) is distributed nearby 3 and scattering of \( \beta \) is very widely in the range of \( 0 < I \leq 1.5 \). As the increasing of Isolation ratio \( I \), \( \beta \) and scattering of \( \beta \) becomes smaller. \( \beta \) is formulized by Isolation ratio \( I \) as follows:

\[
\beta = \frac{s}{I^2} + t \quad \text{(Upper limit is } \beta = u) \tag{2.4}
\]

where \( s, t \) are obtained from the method of least squares of \( \beta \) in the range of \( 1.5 < I \). \( u \) is a simple average of \( \beta \) in the range of \( 0 < I \leq 1.5 \). Fig.2.9 shows the relationship \( s, t, u \) and \( h_{eq} \).

By regression analysis based on Fig. 2.9, each coefficient \( s, t, u \) are formulized by equivalent viscous damping ratio. \( h_{eq} \) is represented in percent on Eq.2.5-2.7.

\[
s = 0.26 h_{eq} + 0.29 \quad \text{(Upper limit is } s = 5.0) \tag{2.5}
\]

\[
t = 0.60 \tag{2.6}
\]

\[
u = 0.09 h_{eq} + 1.28 \quad \text{(Upper limit is } u = 3.0) \tag{2.7}
\]

A solid line of Fig.2.8 shows evaluation of \( \beta \) calculated by Eqs.2.4-2.7. This line is corresponding to moving average \( \beta_{mA} \). Fig.2.10 shows correspondence of prediction value of shear force coefficient calculated by Eq.2.1 and the results of NTHA. The prediction of Eq.2.1 can evaluate properly average value on 4-th story and 10-th story compared with the results of NTHA.
2.4. Variation Correction Factor for Scattering of $\bar{\beta}$

Distribution of $\bar{\beta}$ tends to dispersion for input earthquake and structural characteristics such as damper and superstructure. Then variation correction factor $\nu$ is set for scattering of $\beta$. Variation correction coefficient $\nu$ is set to $\nu_1$ and $\nu_2$ correspond to $\bar{\beta}_{M+1\sigma}$ and $\bar{\beta}_{M+2\sigma}$ using standard deviation $\sigma$ of $\bar{\beta}$. Fig.2.11 shows coefficient of variance $CV$ of $\bar{\beta}$ on each range of Isolation ratio $I$. Based on the data of equivalent viscous damping ratio $h_{eq}$ are 10%-30%. $CV$ is given by as follows:

$$
CV = 0.25 \quad (0 \leq I < 3) \\
CV = -0.06 I + 0.43 \quad (3 \leq I < 6) \\
CV = 0.07 \quad (6 \leq I)
$$

(2.8)

Variation correction factor $\nu_1$ and $\nu_2$ which multiplying for amplification factor $\beta$ is given by as follows:

$$
\nu_1 = (1 + CV) \\
\nu_2 = (1 + 2CV)
$$

(2.9) (2.10)

Fig.2.12 shows correspondence of the prediction of Eq.2.1 considering Eq.2.9 and the results of NTHA of the shear force coefficient on the 4-th and top story. In these cases, almost prediction value of Eq.2.1 evaluates a safe side corresponding to the results of NTHA.

3. ANALYSIS OF HIGHER MODAL RESPONSE

R.I.skinner has revealed the contribution of higher mode is explained by earthquake response of seismically isolated buildings by free-free mode shape vector (skinner et al. 1993). In this study the influence of higher mode, which gives response amplification of shear force coefficient, is revealed by non-linearity modal analysis using free-free mode. By this analysis method, the fluctuation of amplification factor $\beta$ for Isolation ratio $I$ and characteristics of proposed method is explained.

3.1. Modal Analysis with Free-Free Mode Vibration

An appropriate set of mode shapes to represent the response of an isolated structure is set of free-free mode. These mode shapes is obtained when stiffness of isolation story is zero. The $s$-th ($s \leq n$) mode shapes and frequencies for free-free mode vibration are defined by Eq.3.1. where $K_{FF}$ and $M$ are the stiffness and mass matrix of the structure, $s\mathbf{u}_s$ is $s$-th mode shape, $s\omega_s$ is $s$-th natural circular frequency, respectively as the stiffness of isolation story is zero.
Fig. 3.1 shows natural periods and mode shapes vector of base fixed condition mode and free-free mode (normalized \( s \mathbf{u}_{FF} \cdot \mathbf{M} \cdot \mathbf{u}_{FF} = 1 \)). The equation of motion about relative displacement vector \( \mathbf{y} \) becomes as follows:

\[
\mathbf{M} \cdot \ddot{\mathbf{y}} + \mathbf{C} \cdot \dot{\mathbf{y}} + \mathbf{K}_{FF} \cdot \mathbf{y} + (\mathbf{0}_{n-1} \quad F_b) = -\mathbf{M} \cdot \mathbf{1} \cdot \dot{\mathbf{y}}_g
\]

(3.2)

where \( \mathbf{C} \) is damping matrix, \( \mathbf{0} \) and \( \mathbf{1} \) are \( n \)-th zero vector and column vector of which all element are 1. \( F_b \) is the isolator force arising from Bi-linear resistance to displacement. Eq. 3.2 becomes as follows using mode shape vector \( \mathbf{u}_{FF} \):

\[
\ddot{\mathbf{y}} + 2 \cdot s \cdot h \cdot \omega \cdot s \cdot \mathbf{u}_{FF}^T \cdot \mathbf{y} + \mathbf{u}_{FF}^T \cdot \mathbf{M} \cdot \mathbf{u}_{FF} \cdot \mathbf{y} + \frac{\mathbf{u}_{FF}^T \cdot \mathbf{u}_{FF} \cdot F_b}{\mathbf{u}_{FF}^T \cdot \mathbf{M} \cdot \mathbf{u}_{FF}} = -s \cdot \Gamma_{FF} \cdot \mathbf{u}_{FF} \cdot \dot{\mathbf{y}}_g
\]

(3.3)

where \( s \cdot h \) is \( s \)-th damping ratio, \( s \cdot \Gamma_{FF} \) is \( s \)-th participation factor, \( s \cdot \mathbf{u}_{FF} \) is the element above isolation story of \( s \)-th mode shape vector. For the fundamental mode \( (s=1) \), the frequency \( s \cdot \omega = 0 \), the participation factor \( s \cdot \Gamma_{FF} = 1 \). Eq. 3.3 become as follows:

\[
\ddot{\mathbf{y}} + \mathbf{1} \cdot \dot{\mathbf{y}}_g = -\mathbf{F}_b/m_r
\]

(3.4)

where \( m_r \) is the total mass of the structure. For higher modes \((s>1)\), the participation factor \( s \cdot \Gamma_{FF} = 0 \), the equation of motion become as follows:

\[
\ddot{\mathbf{y}} + 2 \cdot s \cdot h \cdot \omega \cdot s \cdot \mathbf{u}_{FF}^T \cdot \mathbf{y} + \mathbf{u}_{FF}^T \cdot \mathbf{M} \cdot \mathbf{u}_{FF} \cdot \mathbf{y} = -s \cdot \mathbf{u}_{FF}^T \cdot \mathbf{u}_{FF} \cdot F_b
\]

(3.5)

From Eq. 3.4 and Eq. 3.5, higher mode response is occurred by the force of isolation story \( F_b \). Time history of \( F_b \) is reflected by input earthquake and characteristics of each isolation device, hence analysis of \( F_b \) is able to evaluate the influence of each mode response. To calculate the maximum response of higher mode using relative acceleration response spectrum for \( F_b/m_r \), relationship between Eq. 3.4 and Eq. 3.5, as follows:

\[
\ddot{\mathbf{y}}_{\text{max}} = \frac{s \cdot \mathbf{u}_{FF}^T \cdot \mathbf{u}_{FF} \cdot m_r \cdot \Gamma_{FF} \cdot S_{a(\text{rel})}(T \cdot h)}{s \cdot \mathbf{u}_{FF}^T \cdot \mathbf{M} \cdot \mathbf{u}_{FF}}
\]

(3.6)

Furthermore, the maximum response of higher modal shear force coefficient by using Eq. 3.5 is calculated by Eq. 3.6, it calculates relative acceleration response. The response of the first mode is including the ground acceleration. Calculation of the maximum response of shear force coefficient by using Square Root of Sum of Square (SRSS) is represented by Eq. 3.7-3.9.

\[
\alpha_1 = \frac{\sum_{i=1}^{N} m_i (\dot{y}_i + \dot{y}_g)}{\sum_{i=1}^{N} m_i} = F_b/m_r \quad (s=1)
\]

(3.7)

\[
s \cdot \alpha_s = \frac{\sum_{i=1}^{N} m_i \cdot \dot{y}_{\text{max}i}}{\sum_{i=1}^{N} m_i} \quad (s \geq 2)
\]

(3.8)

\[
\alpha_{i(SRSS)} = \sqrt{\sum_{s=1}^{N} s \cdot \alpha_s^2}
\]

(3.9)
3.2. Amplification of Shear Force Coefficient

Shear force coefficient of each vibration mode and maximum response of shear force coefficient by using SRSS is calculated by the results of NTHA. Analysis model is same to chapter 2. Type of damper is elasto-plastic hysteresis dampers. The yield displacement of hysteresis damper $\delta_y$ is fixed at 1.0 cm, yield shear coefficient of dampers $\alpha$ is fixed at 0.04. Fig.3.2 shows relative acceleration response spectrum $S_{\alpha(t)}$ for $F_i/m_i$. Relative acceleration response spectrum shows that the peak-period corresponds to the seismic isolation period 4.0sec. Further, Short-period range near the natural period of free-free mode vibration in Fig.3.1, response spectrum is amplified. This range directly affects to the higher mode response.  

![Figure 3.1. Mode shape vector](image)

![Figure 3.2. Relative acceleration response spectrum](image)

Fig.3.3 shows shear force distribution of SRSS (considering 5th mode), the results of NTHA, the prediction of Japanese design code Eq.1.5 and proposed method Eq.2.1. SRSS is corresponding to NTHA (i.e. each mode response evaluated by Eq.3.4 and Eq.3.6 is valid for response characteristics of SI-buildings). Further, proposed method Eq.2.1 is corresponding to the results of NTHA. Fig.3.4 shows variation of each mode response for the fluctuation of Isolation ratio $I$. In the cases of $I \leq 3.0$, the influence of higher mode is more amplified. In this range, Eq.1.5 underestimated earthquake response. The proposed method Eq.2.1 represents adequately the amplification of shear force coefficient caused by higher mode response.

![Figure 3.3. Shear force coefficient distribution of each mode](image)

![Figure 3.4. Variation of each mode response for the fluctuation of Isolation ratio $I$](image)

4. EFFECTS OF AMPLIFICATION FACTOR

Evaluation methods of shear force coefficient and amplification factor are determined by seismically isolated buildings with Bi-linear restoring characteristics. The characteristics of isolation layer are not necessarily Bi-linear restoring characteristics. Therefore we propose a method of obtaining Bi-Linear approximation for various restoring force model such as Tri-Linear or Ramberg-Osgood characteristics. Hence the proposed method Eq.2.1 can be adapted to various kinds of SI-buildings. And it is confirmed that the evaluation method of shear force coefficient is sufficient to evaluate the Overturning Moment.
4.1. Adaption for Another Characteristics Model

The characteristics of isolation layer is not Bi-linear restoring characteristics, a method of obtaining Bi-Linear approximation for various restoring force model such as Tri-Linear or Ramberg-Osgood characteristics is set as follows:

1. Second stiffness $k_2$ of Bi-linear (Bi_eq) is set tangent stiffness of the original model in the maximum displacement $\delta_{\text{max}}$.

2. Initial stiffness $k_1$ of Bi-linear (Bi_eq) is determined as equivalent viscous damping factor $\zeta_{\text{eq}}$ is equal to the original model in the maximum displacement $\delta_{\text{max}}$.

Fig.4.1 shows approximation method for Tri-linear model and Ramberg-Osgood model.

To confirm that evaluation of shear force coefficient using above method is effective, NTHA carried out on 10-sotry mass shear model with Tri-linear and R-O model characteristics for isolation layer. Parameters of NTHA are shown in table 4.1. Before and After approximation, shear force coefficient of 10-th story is calculated by Eq.2.1. Before approximation, Tri-linear model is initial stiffness and R-O model is Tangent stiffness of starting point for evaluation of isolation ratio $I$, respectively. Then prediction value of Eq.2.1 evaluates fairly on the safe side shown is Fig.4.2. After approximation, On the other hand, approximation method for Bi-linear model is approaching the appropriate evaluation.

Table 4.1. Characteristics of isolation story for analysis model

<table>
<thead>
<tr>
<th>Original Tri-linear</th>
<th>Bi-linear (eq)</th>
<th>Original R-O</th>
<th>Bi-linear (eq)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\delta_1$[cm]</td>
<td>$\alpha_1$[%]</td>
<td>$\delta_2$[cm]</td>
<td>$\alpha_2$[%]</td>
</tr>
<tr>
<td>0.1</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>1.0</td>
<td>1.2</td>
<td>1.8</td>
<td>1.8</td>
</tr>
<tr>
<td>0.1</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>1.0</td>
<td>2.0</td>
<td>2.0</td>
<td>2.0</td>
</tr>
<tr>
<td>1.0</td>
<td>3.0</td>
<td>3.0</td>
<td>3.0</td>
</tr>
</tbody>
</table>

Figure 4.1. Approximation methods for Tri-linear model and Ramberg-Osgood model

Figure 4.2. The evaluation of shear force coefficient considering approximation method
4.2. Evaluation of Overturning Moment

It is necessary for high-rise SI-buildings to be careful with uplift of Isolator caused by Overturning Moment. Analysis model is same to chapter 2. Overturning Moment on the isolation story is calculated by as follows:

\[ OTM_i = \sum_{i=1}^{N_t} W_i \cdot \alpha_i \cdot H_i \]  

(4.1)

Where \( \alpha_i \) is calculated by Eq. 1.5 or Eq.2.1, \( N_t \) is the top story. Fig.4.3 shows comparison overturning moment by using each evaluation method and that of NTHA. Proposed method introducing \( \beta_i \) is also sufficient to evaluate of Overturning Moments.

![Figure 4.3. Corresponding to the results of NTHA (Overturning Moment on the isolation story)](image)

5. CONCLUSION

This paper proposed an evaluation method of shear force coefficient for SI-buildings. The results obtained by analytical studies are as follows; 1) It is confirmed that the prediction of the proposed method can evaluate properly or safe side corresponding to the results of NTHA. 2) Response amplification of shear force coefficient is explained by non-linearity modal analysis with free-free mode vibration. 3) We proposed an approximation method Tri-linear and Ramberg-osgood into Bi-linear. By using this approximation, the proposed method is adopted to various kinds of SI-buildings. 4) Proposed method of shear force coefficient is sufficient to evaluation of overturning moment. On these studies, proposed methods are corresponding to results of NTHA.

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