INVESTIGATING THE EFFECT OF SOIL-STRUCTURE INTERACTION ON COLLAPSE OF INTEGRAL ABUTMENT BRIDGES

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Abstract

In this study, a performance based approach is employed to investigate the effect of soil-structure interaction (SSI) on the response of integral abutment bridges. Non-linear Incremental Dynamic Analysis (IDA) is used for the analysis the structure and to generate fragility curves for this type of bridge. Finite element models are constructed for three types of integral abutment bridges. Simulation are performed considering and neglecting the SSI effects. A total of 20 selected ground motions was used in the IDA of the bridge models. The spectral acceleration of ground motions is chosen as the Intensity Measure (IM) and relative displacement and drift of the abutment back walls and pier columns are chosen as engineering demand parameters (EDPs). The Collapse Margin Ratio (CMR) is chosen as a measure to objectively compare the response of non-SSI and SSI models. In the cases studied here, the SSI models showed significant reduction in collapse margin ratio when compared to their counterpart non-SSI model. This indicates that ignoring the soil effect may result in under estimation of the probability of collapse.

Keywords: Integral Abutment Bridges; Soil Structure Interaction (SSI); Incremental Dynamic Analysis (IDA); Collapse Margin Ratio (CMR); Probability of Collapse.
1. Introduction

Several bridges have collapsed worldwide as a result of strong earthquakes (e.g. [1]). Lack of understanding of soil-structure interaction (SSI) is one of several reasons for such collapses. As discussed by [1] and [2], soil-pile-structure interaction has led to damage and rupture of bridge piers in Northridge Earthquake in 1994. Collapse of the 630-meter long Hanshin Expressway Bridge during the Kobe Earthquake in 1995 is another example of the contribution of SSI [18]. An analytical study of the Hanshin Expressway showed that soil characteristics had a significant effect on the behaviour of this structure [3]. The inertial SSI effect also had led to elongation of bridge’s fundamental periods resulting in a more severe response. Similar phenomenon was observed in the collapse of the Cypress Bridge in Oakland during the Loma Prieta earthquake in 1989 [4].

Current seismic design codes are lacking comprehensive provisions to deal with the SSI effects [4]. In the National Building Code of Canada (NBCC2015) [5], the effect of soft soil on the response spectrum is incorporated via the $F_\alpha$ and $F_\nu$ site response coefficients.

However, the contribution of SSI effects on the structural response needs to be effectively addressed in the next generation of bridge design codes to minimize seismic risk and avoid catastrophic failure of bridges.

In this study, the Incremental Dynamic Analysis (IDA) method is used as part of a methodology developed to investigate the effects of soil-structure interaction on the potential collapse of integral abutment bridges, within a performance-based framework. The Collapse Marginal Ratios (CMR) introduced by FEMA P-695 [6] as a collapse measure for buildings is adopted here to bridge structures. CMR is used to compare the performance of SSI and non-SSI models. Further information on the methodology and approach used for the study conducted by the authors can be found in [7].

2. Methodology

The proposed methodology consists of three steps:

- **Step 1**: Model construction and analysis,
- **Step 2**: Calculation of probability of collapse,
- **Step 3**: Interpretation and comparison of collapse margin ratios.

An incremental dynamic analysis is performed using a set of selected ground motions on detailed FE models of the bridges. Probability of collapse is then calculated from the IDA analysis outputs and used to construct the fragility curves. The collapse margin ratios (CMR) are then predicted for each set of analysis. A schematic of the methodology adopted here is presented in Fig.1.

![Fig. 1 – General framework of the study research](image)

### 2.1 Model Construction and Analysis (Step 1)

In this step, a detailed finite element model of each of the bridge archetypes is developed. A set of suitable ground motions is selected and the IDA analysis is performed on the models. Details of model construction and analysis is described below.
2.1a FE Model construction

The SeismoStruct software is employed to develop the 3-D nonlinear models of integral abutment [8]. Geometric non-linearity, as well as, non-linear behaviour of primary structural elements is considered in the model.

Two archetype models are developed for each bridge: one considering the SSI effects and another one without considering it. In the SSI models, the effect of soil-structure interaction from soils supporting the piles and abutment backwalls are taken into account. The SSI p-y link introduced by is employed here to simulate the effect of soil surrounding piles. The properties of soil sub-layer models are considered in determining the link properties. The effect of the soil behind the abutment backwall is simulated using springs as recommend by the CALTRANS guidelines [9].

The p-y links include features such as loading and unloading rules, modeling of radiation damping and cyclic degradation, and slack zone for the analysis of shallow and deep foundation [10]. The properties of p-y link are determined based on properties of soil according to API code recommendations for load-deflection (p-y) curves for Sand and soft clay [11] and L-Pile reference manual [12] for stiff clay layer based on Reese model for stiff clay without water.

2.1b Ground motions

FEMA P-695 [6] recommendations are employed to select a set of 20 selected ground motions from the PEER Strong Motion database.

According to FEMA P-695, ground motions whose response spectrum best fits to mean site spectrum are chosen. The selected ground motions were applied to the models without any modification or scaling at the period of vibration of the bridges in the analysis. To produce the fragility curves using Incremental Dynamic Analysis (IDA), intensity of ground motions is increased until structural failure occurs. Period of interest is considered the range between smallest and biggest period of the bridge models. Response spectra for five out of the twenty selected ground motions and the target spectrum along with the period of interest are shown in Fig. 2. Vancouver’s Uniform Hazard Spectrum (UHS) is also shown in the figure for comparison. To estimate UHS, Vs30 is considered based on NBCC 2005 soil site class C, with an average shear wave velocity of 360-750 m/s.

![Ground motion spectra](image-url)

**Fig. 2** – Ground motion spectra along with the target spectrum (2% in 50 years and 5% damping), and the range of period of interest

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2.1c Incremental Dynamic Analysis (IDA)

In the Incremental Dynamic Analysis method (IDA), the structure is subjected to a sequence of excitations of increasing intensity. The analysis is continued until the structure reaches a pre-defined collapse state. The median collapse intensity can be extracted from IDA analysis [14]. Collapse conditions and the collapse fragility curves are then calculated for each archetype model using a cumulative distribution function (CDF).

In this study, a coefficient of 0.25 was used as the initial multiplier and further increments are performed using a 0.25 increase at each stage. Acceleration time histories are applied in directions transverse and perpendicular to the bridge decks.

Spectral acceleration is chosen as the intensity measure (IM) here and drifts of the abutment backwalls and pier columns are considered as engineering demand parameters (EDPs). The main objective here is the evaluation of performance of the seismic-force resisting system. To achieve this purpose, performance criteria are evaluated based on ‘collapse prevention’ according to FEMA P-695.

In this study, the event of collapse is not simulated explicitly; instead, non-simulated collapse modes are evaluated using limit state checks on structural response quantities calculated in the analyses (performance criteria).

In non-simulated collapse modes, failure of the first primary structural component is assumed as an indicator of structural collapse. As discussed in FEMA P-695, non-simulated limit state checks may result in lower estimates of the median collapse compared to the case that all the local failure modes are directly simulated [6].

2.2. Probability of Collapse (Step 2)

Fragility curves are calculated based on the results of the IDA analyses. Probability of collapse is calculated as a function of spectral acceleration ($S_a$). As suggested by [15], a log-normal cumulative distribution function is fitted to data using least square method.

This approach can be used to fit fragility functions for a variety of situations, particularly, collapse fragility functions obtained from structural analysis data. A CDF is fitted to IDA data, to provide a continuous estimate of the probability of collapse as a function of $S_a$ using the following equation:

$$P(C | S_a = x) = \Phi \left( \frac{\ln(x / \mu)}{\sigma} \right)$$

where,

$P(C | S_a = x)$ is the probability that a ground motion with $S_a = x$ will cause the structure to collapse,

$\Phi(\cdot)$ is the standard normal cumulative distribution function (CDF),

$\mu$ is the median of the fragility function

and $\sigma$ is the standard deviation of $\ln S_a$ or dispersion of $S_a$.

2.3. Collapse Marginal Ratio (CMR) (Step 3)

Collapse Margin Ratio (CMR) is defined as the ratio between the median collapse intensity ($S_{CT}$) and the Maximum Considered Event (MCE) intensity ($S_{MT}$). MCE intensity ($S_{MT}$) is obtained from the response spectrum of MCE ground motions at the fundamental period, $T_1$ [6]. $CMR$ is calculated as per Eq. 2 below.

$$CMR = \frac{S_{CT}}{S_{MT}(T_1)}$$
The CMR parameter offers an objective indicator for structure collapse. This parameter combines the fragility curve with the site-specific response spectrum. Higher CMR values indicate less chance of failure while cases with lower CMR values have a higher probability of failure.

3. Bridge Models

In this section a brief discussion of the bridge models selected for this study is provided. Details of how the SSI effects have been incorporated into these models are also presented.

3.1. Archetype Models

Three different types of bridges are considered to calculate and compare the probability of collapse of the integral abutment bridges (see Table 1 for details). Two archetype models namely, SSI and non-SSI models, are defined for each bridge type.

M₁ model corresponds to a simplified model of a single span integral abutment bridge with a 30° skew angle. Fixed support was applied to the base of the foundations of M₁ and all the other non-SSI models. M₂ model is the SSI version of the M₁ model considering effect of soil around the abutment piles assuming a soil sub-layer arrangement along the abutment piles. SSI effects behind the abutment backwall and around the piles are simulated by employing CALTRANS springs [9] and SSI p-y links corresponding to each layer of soil as discussed in section 2.1. In the SSI models, seismic loading was applied to the base of piles. In the non-SSI models, seismic loading was applied to the base of abutment and pier foundations.

M₃ and M₄ models correspond to non-SSI and SSI models of a three-span integral abutment bridge with a 15° skew angle. Finally, M₅ and M₆ models are the non-SSI and SSI models of a three-span two-band common semi-integral abutment bridge with a 6° skew angle. Fig. 3 shows the schematics of three type bridges studied here.

Fig. 3 – Shows different three type of bridges used in this study
Table 1 – Summary of the bridge types and archetype models are developed and used in this study

<table>
<thead>
<tr>
<th>Archetype Model</th>
<th>Structural Feature</th>
<th>Span Length (m)</th>
<th>Skewness Angle (°)</th>
<th>Horizontal Radius (m)</th>
<th>SSI Feature</th>
</tr>
</thead>
<tbody>
<tr>
<td>M₂</td>
<td>Integral abutment bridge with abutment pile foundations and total 9 pre-stressed precast girders</td>
<td>38</td>
<td>30°</td>
<td>–</td>
<td>CALTRANS springs and SSI p-y links</td>
</tr>
<tr>
<td>M₄</td>
<td>Integral abutment bridge with pier &amp; abutment pile foundations and total 4 pre-stressed precast girders</td>
<td>17-29-19</td>
<td>15°</td>
<td>1009.95</td>
<td>CALTRANS springs and SSI p-y links</td>
</tr>
<tr>
<td>M₆</td>
<td>Two-band semi-Integral abutment bridge with common seat abutment, pier pilecap deep foundation, intermediate diaphragm, and total 8 pre-stressed precast girders</td>
<td>27.5-37.5-27.5</td>
<td>6°</td>
<td>–</td>
<td>CALTRANS springs</td>
</tr>
</tbody>
</table>

The soil sub-layer arrangement shown in Fig. 4 is considered to account for soil effect around the abutment and pier piles.

Fig. 4 – Shows assumed soil sub-layer around the piles and location of the assigned CALTRANS springs and SSI p-y links in the SSI models

3.2. SSI model parameters

The sub-layer arrangement as shown in Fig. 5 is based on a geotechnical recommendation report provided by the Hatch Mott MacDonald and MMM Group (H5M joint venture) for the design stage of one of the newly constructed bridges as a part of the Port Mann Highway 1 Project in Vancouver [16].

The API code and L-Pile software manual are employed to develop the p-y curves for clay layers and sand layer, respectively (shown in Fig. 5). The parameters used in SeismoStruct are extracted from the p-y curves. Parameters of the tri-linear backbone p-y curves for each layer are presented in Table 2.
For simplicity, soil layers around the abutment and pier piles are considered to be identical in M₂ and M₄ models.

Table 2 – Calculated parameters which obtained fitting a tri-linear curve to the derived p-y curves

<table>
<thead>
<tr>
<th>Layer</th>
<th>Thickness (m)</th>
<th>Depth (m)</th>
<th>$k_0$  (kN/m)</th>
<th>$\alpha$</th>
<th>$B$</th>
<th>$P_u$ (kN)</th>
<th>$F_c$ (kN)</th>
<th>$F_y$ (kN)</th>
<th>$\beta_n$</th>
<th>$P_u/F_y$</th>
<th>$F_c/F_y$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stiff Clay</td>
<td>2</td>
<td>1</td>
<td>11490.6</td>
<td>0.0969</td>
<td>0.040</td>
<td>117.2</td>
<td>98.6</td>
<td>1.189</td>
<td>0.595</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soft Clay</td>
<td>2</td>
<td>3</td>
<td>15651.5</td>
<td>0.1313</td>
<td>0.051</td>
<td>289.3</td>
<td>208.3</td>
<td>1.389</td>
<td>0.458</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soft Clay</td>
<td>2</td>
<td>5</td>
<td>15814.1</td>
<td>0.1313</td>
<td>0.051</td>
<td>292.3</td>
<td>210.5</td>
<td>1.389</td>
<td>0.458</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soft Clay</td>
<td>3</td>
<td>7.5</td>
<td>20756.1</td>
<td>0.1313</td>
<td>0.051</td>
<td>383.7</td>
<td>276.2</td>
<td>1.389</td>
<td>0.458</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silty Sand</td>
<td>3.7</td>
<td>10.85</td>
<td>685185.2</td>
<td>0.3484</td>
<td>0.060</td>
<td>27624.2</td>
<td>25900</td>
<td>1.067</td>
<td>0.714</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

in which,

$k_0$: initial stiffness

$\alpha$: second segment coefficient of stiffness of the nonlinear dynamic soil structure interaction model

$\beta$: stiffness ratio parameter in the SSI model which defines the stiffness of the third segment in proportion to $k_0$

$P_u$: soil ultimate strength

$F_c$: soil strength ratio at first turning point

$F_y$: soil yield strength

$\beta_n$: strength ratio parameter in the SSI model

In this study, to account for the effect of embankment soil behind the abutment backwalls, CALTRANS springs are used in the M₂, M₄ and M₆ models. Stiffness of the abutment due to passive pressure of backfilling behind it is obtained based on section 7.8.1-2 SDC 2013 [9] as per Eq. 3 and summarized in Table 3.

$$k_{abut} = k_i \left( \frac{h}{1.7m} \right)$$

where, $k_i$ is the initial stiffness of the embankment fill material behind the abutments, and based on section 7.81-1 of SDC 2013 can be calculated using Eq. 4:
\[ k_i \approx \frac{28.7 \text{kN/mm}}{m} \]  

(4)

in which,

\( w \) is the projected width of the backwall or diaphragm for seat and diaphragm abutments respectively.

\( h \) is the effective height of abutment which is determined based on the diaphragm design.

Here, the effective height is chosen assuming that abutment diaphragms are designed for full soil pressure.

Table 3 – Summary of calculated stiffness of the bridge abutments due to the embankment passive pressure force resisting movement

<table>
<thead>
<tr>
<th>Models</th>
<th>( k_i ) (kN/mm/m)</th>
<th>( w ) (m)</th>
<th>( k ) (m)</th>
<th>( k_{abut.} ) (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M2</td>
<td>28.7</td>
<td>19.2</td>
<td>4.33</td>
<td>1404</td>
</tr>
<tr>
<td>M4</td>
<td>28.7</td>
<td>13.69</td>
<td>5.1</td>
<td>1178.54</td>
</tr>
<tr>
<td>M6</td>
<td>28.7</td>
<td>34.88</td>
<td>4</td>
<td>2355.43</td>
</tr>
</tbody>
</table>

4. Results

4.1. IDA Results

The maximum pier column relative displacements (displacement between top of pier footing and top of the pier columns) were extracted from IDA simulations at the collapse stage. Fig 6. Shows these relative displacements for M4 model. Hysteresis loops corresponding to moment vs curvature and base shear vs drift behaviors are also calculated for pier column members. Figure 7 shows the pier column total drift ratio of the M3 and M4 models versus total base shear and moment for the Chi Chi ground motion. These graphs correspond to IDA simulations that have resulted in collapse. The M3 curve shows a collapse at a time-history scale factor of 2.0 and the M4 model shows collapse at scale factor of 0.75.

![Graph showing predicted pier column relative displacement](image)

Fig. 6. Predicted pier column relative displacement along the bridge deck for M4 model.
As shown in Fig. 7, the column’s drift for the SSI model (M4 model) is significantly reduced at the collapse level. Despite of this reduction in column drift, as will be discussed later, probability of collapse is increased in the SSI model for a given earthquake. This is due to the change in failure mode in the SSI models. Fig. 8 shows a summary of the failure modes observed in various bridges based on the IDA simulations. As an example, in the M1 model, failure is mainly due to abutment shear force whereas in its SSI counterpart (M2 Model), the majority of failures are due to girder shear force.

Fig. 7 – Pier column drift ratio across the bridge deck versus (a) total base shear, and (b) total moment calculated using IDA - M3 and M4 models.

4.2. Probability of Collapse and Collapse Marginal Ratio (CMR) Results

The MATLAB code developed by Baker [17] was employed to calculate the fragility curves. Calculated probabilities of collapse of the archetype models are presented in Fig. 9. Assuming that the shear wave velocity of the top 30m of the site sub-layer soil is between 360m/s and 750m/s, which is equivalent to soil site class C based on [5], collapse margin ratios for 10%, 20%, and 50% (median) were calculated and presented in Table 4.

Although pier drifts predicted in the SSI models (as shown in Fig. 7) are typically smaller than the corresponding non-SSI models, redistribution of load to other components, such as the abutment, has led to
earlier structural failure. This has consequently resulted in shifting the CMR curve corresponding to SSI models to the left when compared with the non-SSI counterparts.

As shown in Fig. 9, the SSI model exhibits a reduction in predicted CMR values. As a result, for a given $S_a$, probability of collapse is increased. For instance, the M3 model subjected to the Chi Chi earthquake fails at a $S_a$ level of 1.61g due to shear force in the abutments, whereas the SSI model of the same bridge (M4 model) has failed at $S_a$ of 0.54g due to Abutment concrete confinement strain and steel strain. As shown in Fig 10, reduction of CMR in the SSI model has led to an increase in probability of collapse for a given $S_a$.

![Fig. 9 – Calculated probability of collapse for all the archetype models](image)

![Fig. 10 – Reduction of CMR and increase of Probability of Collapse in SSI model](image)
Table 4 – Summary of the calculated collapse margin ratios for median (50%), 10%, and 20% and for all models

<table>
<thead>
<tr>
<th>Model</th>
<th>( T_1 ) 10% (sec)</th>
<th>( S_c ) 10% (g)</th>
<th>( S_c ) 20% (g)</th>
<th>( S_c ) @ Median (g)</th>
<th>( S_MT ) @ 10% (g)</th>
<th>CMR 10%</th>
<th>CMR 20%</th>
<th>CMR Median</th>
</tr>
</thead>
<tbody>
<tr>
<td>M_1</td>
<td>0.25</td>
<td>2.59</td>
<td>2.86</td>
<td>3.46</td>
<td>0.90</td>
<td>2.88</td>
<td>3.18</td>
<td>3.84</td>
</tr>
<tr>
<td>M_2</td>
<td>0.28</td>
<td>1.19</td>
<td>1.33</td>
<td>1.67</td>
<td>0.86</td>
<td>1.39</td>
<td>1.56</td>
<td>1.95</td>
</tr>
<tr>
<td>M_3</td>
<td>0.34</td>
<td>1.13</td>
<td>1.34</td>
<td>1.90</td>
<td>0.80</td>
<td>1.41</td>
<td>1.68</td>
<td>2.38</td>
</tr>
<tr>
<td>M_4</td>
<td>0.37</td>
<td>0.94</td>
<td>1.08</td>
<td>1.39</td>
<td>0.76</td>
<td>1.24</td>
<td>1.42</td>
<td>1.84</td>
</tr>
<tr>
<td>M_5</td>
<td>0.37</td>
<td>1.21</td>
<td>1.45</td>
<td>2.08</td>
<td>0.76</td>
<td>1.60</td>
<td>1.91</td>
<td>2.74</td>
</tr>
<tr>
<td>M_6</td>
<td>0.37</td>
<td>1.11</td>
<td>1.35</td>
<td>1.99</td>
<td>0.76</td>
<td>1.46</td>
<td>1.78</td>
<td>2.62</td>
</tr>
</tbody>
</table>

5. Conclusions

In this study, Incremental Dynamic Analysis (IDA) within the framework of Performance-based Design was adopted to simulate the response of three bridge models. In all cases, the analyses were carried out with and without considering the SSI effects. A set of 20 selected ground motions were chosen for the IDA analysis.

FEMA P695’s non-simulated collapse mode concept was adopted as a collapse indicator. Fragility curves were extracted based on predictions of IDA simulations for both non-SSI and SSI models. The concept of Collapse Margin Ratio (CMR) was employed here to compare the performance of these models. It was shown that the SSI models typically collapsed at lower \( S_c \) levels. As a result, the collapse margin ratio values for the SSI models were smaller than their non-SSI counterparts.

This has led to a shift in fragility curves of SSI models to the left side of the non-SSI curves. It was shown in this study that for the soil condition considered here, neglecting the soil-structure interaction leads to under prediction of the probability of collapse and thus, results in a non-conservative design.

The approach presented in this study offers an insight into the response of structure-soil system subjected to severe failure loading. Such an approach can be employed in practice to adjust the non-SSI fragility curves to reflect the SSI effects.

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8. References


