Performance-Based Analysis and Design for California Ordinary Standard Bridges

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and

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Caltrans

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Outline

- Testbed California Ordinary Standard Bridges (OSBs) and Computational Models
- PEER PBEE Assessment Methodology
- Parametric Probabilistic Seismic Performance Assessment Framework
- Simplified Risk-Targeted Performance-Based Seismic Design (PBSD) Method
- Concluding Remarks & Recommendations for Future Work
## Ordinary Standard Bridge (OSB) Testbeds Considered

<table>
<thead>
<tr>
<th>Bridge Designation</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>MAOC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Name</td>
<td>Jack Tone Road Overcrossing</td>
<td>La Veta Avenue Overcrossing</td>
<td>Jack Tone Road Overhead</td>
<td>Massachusetts Avenue Overcrossing</td>
</tr>
<tr>
<td>Location: City, State</td>
<td>Ripon, CA</td>
<td>Tustin, CA</td>
<td>Ripon, CA</td>
<td>San Bernardino, CA</td>
</tr>
<tr>
<td>Total Length</td>
<td>220.4 ft</td>
<td>299.8 ft</td>
<td>418.2 ft</td>
<td>413.4 ft</td>
</tr>
<tr>
<td>Number of Spans and Span Length</td>
<td>2 Span 1: 108.6 ft Span 2: 111.8 ft</td>
<td>2 Span 1: 154.8 ft Span 2: 145 ft</td>
<td>3 Span 1: 156.2 ft Span 2: 144 ft Span 3: 118 ft</td>
<td>5 Span 1: 49.2 ft Span 2: 94.5 ft Span 3: 91.9 ft Span 4: 99.7 ft Span 5: 78.1 ft</td>
</tr>
<tr>
<td>Type of Column Bent</td>
<td>Single Column (RC Circular) Column Diameter: 5.5 ft Column Height: 19.7 ft</td>
<td>Two-column (RC Circular) Column Diameter: 5.5 ft Column Height: 22.0 ft</td>
<td>Three-column (RC Circular) Column Diameter: 5.5 ft Column Height: 24.6 ft</td>
<td>Four-column (RC Circular) Column Diameter: 4.0 ft Column Heights: 29.5 ft, 31.5 ft, 30.7 ft, 27.4 ft</td>
</tr>
<tr>
<td>Skew</td>
<td>33 degrees</td>
<td>0 degrees</td>
<td>36 degrees</td>
<td>8 degrees</td>
</tr>
</tbody>
</table>
Computational Bridge Models

Schematic Representation of FE Model of Bridge B in OpenSees:

- Confined concrete material hysteresis
- Unconfined concrete material hysteresis
- Reinforcing steel material hysteresis

- Fiber-section Euler-Bernoulli force-based beam-column element
- Elastic beam-column element
- Rigid beam-column element

- Abutment-backfill interaction
- Horizontally coupled elastomeric bearing element
- Exterior shear key spring
- Backfill uniaxial springs with linear variation (proportional to skew) of strengths and initial stiffnesses

Graphs showing material properties:
- Stress-strain curves for confined and unconfined concrete, reinforcing steel.
- Elastic modulus, strain at yield, yield strength.

Diagram showing:
- Bridge structure with elements and interactions:
  - Beam-column elements
  - Stiffness and strength variations
  - Abutment and backfill integration
  - Shear key interaction
PEER Performance-based Earthquake Engineering Assessment Methodology

Hazard analysis
- Hazard model \( P[IM|L] \)
- Site hazard \( v(IM) \)
- IM: intensity measure

Demand analysis
- Structural model \( P[EDP|IM] \)
- Structural response \( v(EDP) \)
- EDP: engineering demand param

Damage analysis
- Fragility function \( P[DM|EDP] \)
- Damage \( v(DM) \)
- DM: damage measure

Loss analysis
- Loss model \( P[DV|DM] \)
- Performance \( v(DV) \)
- DV: decision variable

Bridge

L: Location
D: Design

L, D

<table>
<thead>
<tr>
<th>IM: intensity measure</th>
<th>EDP: engineering demand param</th>
<th>DM: damage measure</th>
<th>DV: decision variable</th>
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<td>IM: intensity measure</td>
<td>EDP: engineering demand param</td>
<td>DM: damage measure</td>
<td>DV: decision variable</td>
</tr>
</tbody>
</table>

\( P(X | Y) \): Conditional PDF of X given Y

\( \nu_X(x) \): Mean annual rate of X exceeding the threshold value x

\( G(X | Y) \): Conditional complementary CDF of X given Y

(Adapted from Porter 2003)

How likely is an event of intensity IM, for this location?

What engineering demands (force, deformation, etc.) will this bridge experience?

What physical damage will this bridge experience?

What loss (deaths, dollars, and downtime) will this bridge experience?

Is the performance of the bridge acceptable?

\[
\nu_{DV} = \int \int G_{DV|DM} (dv | DM = dm) \cdot dG_{DM|EDP=edp} (dm | EDP = edp) \cdot dG_{EDP|IM=im} (edp | IM = im) \cdot dv_{IM} (im)
\]

\( P(X | Y) \): Conditional PDF of X given Y

\( \nu_X(x) \): Mean annual rate of X exceeding the threshold value x

\( G(X | Y) \): Conditional complementary CDF of X given Y

\[ IM : \quad S_{a, \text{avg}}(T_1, \ldots, T_n) = \left[ \prod_{k=1}^{n} S_a(T_k) \right]^{1/n} \]

Kohrangi, Bazzurro and Vamvatsikos (2016)

\[ V_{S_a, \text{avg}}(x) = \sum_{s=1}^{N_a} V_{\text{scenario}, s} \cdot P \left[ \prod_{p=1}^{n} S_{a}(T_p) \right]^{1/n} > x \mid \text{scenario}_s \]

Seismic Hazard Curves

Averaging period range: \( T_{1, \text{trans}} - 2.5 T_{1, \text{trans}} \)

- \( RP = 72 \text{ yrs} \) (50% PE in 50 years)
- \( RP = 224 \text{ yrs} \) (20% PE in 50 years)
- \( RP = 475 \text{ yrs} \) (10% PE in 50 years)
- \( RP = 975 \text{ yrs} \) (3% PE in 50 years)
- \( RP = 2475 \text{ yrs} \) (2% PE in 50 years)
- \( RP = 4975 \text{ yrs} \) (1% PE in 50 years)
Selection of Ensembles of Site-specific Risk-consistent Ground Motion Records

5% damped pseudo accel. response spectra, mean RP of 1M exceedance = 72 yrs.

$S_{n_{avg}} = 0.388 \text{ g}$

$T [s]$  $10^{-1}$  $10^{0}$

Conditional median spectrum given $S_{n_{avg}}$
- 2.5 and 97.5 percentile target conditional spectra
- UHS
Response spectra of selected ground motions

5% damped pseudo accel. response spectra, mean RP of 1M exceedance = 224 yrs.

$S_{n_{avg}} = 0.586 \text{ g}$

$T [s]$  $10^{-1}$  $10^{0}$

Conditional median spectrum given $S_{n_{avg}}$
- 2.5 and 97.5 percentile target conditional spectra
- UHS
Response spectra of selected ground motions

5% damped pseudo accel. response spectra, mean RP of 1M exceedance = 475 yrs.

$S_{n_{avg}} = 0.735 \text{ g}$

$T [s]$  $10^{-1}$  $10^{0}$

Conditional median spectrum given $S_{n_{avg}}$
- 2.5 and 97.5 percentile target conditional spectra
- UHS
Response spectra of selected ground motions

5% damped pseudo accel. response spectra, mean RP of 1M exceedance = 975 yrs.

$S_{n_{avg}} = 0.893 \text{ g}$

$T [s]$  $10^{-1}$  $10^{0}$

Conditional median spectrum given $S_{n_{avg}}$
- 2.5 and 97.5 percentile target conditional spectra
- UHS
Response spectra of selected ground motions

5% damped pseudo accel. response spectra, mean RP of 1M exceedance = 2475 yrs.

$S_{n_{avg}} = 1.13 \text{ g}$

$T [s]$  $10^{-1}$  $10^{0}$

Conditional median spectrum given $S_{n_{avg}}$
- 2.5 and 97.5 percentile target conditional spectra
- UHS
Response spectra of selected ground motions

5% damped pseudo accel. response spectra, mean RP of 1M exceedance = 4975 yrs.

$S_{n_{avg}} = 1.32 \text{ g}$

$T [s]$  $10^{-1}$  $10^{0}$

Conditional median spectrum given $S_{n_{avg}}$
- 2.5 and 97.5 percentile target conditional spectra
- UHS
Response spectra of selected ground motions

Refs.: Baker and Jayaram (2011)
Kohangi, Bazzurro, Vamvatsikos, and Spillatura (2017)
### Definition of Limit-States and Associated Engineering Demand Parameters

<table>
<thead>
<tr>
<th>Limit-state ((LS))</th>
<th>Associated Engineering Demand Parameter ((EDP))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete cover crushing ((LS_1))</td>
<td>Maximum absolute compressive strain of any longitudinal rebar in any column: (EDP = \max\left(\max\left(\max e^\text{bar}_{\text{comp}}(t)\right)\right))</td>
</tr>
<tr>
<td>Longitudinal buckling ((LS_2))</td>
<td>Maximum tensile strain of longitudinal rebar in any column: (EDP = \max\left(\max e^\text{tensile}(t)\right))</td>
</tr>
<tr>
<td>Longitudinal fracture ((LS_3))</td>
<td>Maximum difference of tensile (positive) and compressive (negative) strain, the latter following the former, of any longitudinal rebar in any column: (EDP = \max\left(\max \left</td>
</tr>
<tr>
<td>Shear key damage ((LS_4))</td>
<td>Maximum horizontal displacement of any shear key normalized by the displacement at peak strength: (EDP = \max\left(\max \left</td>
</tr>
</tbody>
</table>

“Structural displacements, which can be directly related to damage potential through material strains (structural damage)…, are [currently] checked through coarse and unreliable methods…”

- Nigel Priestley, 2007

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- Nigel Priestley, 2007
\[ \nu_{EDP_k}(\delta) = \int_{IM} P[EDP_k > \delta | IM = x] \cdot d\nu_{IM}(x) \]

\[ \nu_{IM}(x + dx) - \nu_{IM}(x) \]

RP = 72 yrs
RP = 975 yrs
RP = 4975 yrs

Probabilistic Seismic Demand Hazard Analysis

IM Deaggregation of Demand Hazard

Demand Hazard Curve

RP = 72 yrs
RP = 975 yrs
RP = 4975 yrs
Probabilistic Seismic Demand and Capacity

Probability density function of $EDP_k$:

$$f_{EDP_k}(\delta) = \frac{d}{d\delta} \left( 1 - \frac{v_{EDP_k}(\delta)}{v_{IM}(x = 0)} \cdot \frac{F_{EDP_k}(\delta)}{F_{EDP_k}(\delta) - F_{EDP_k}(0)} \right)$$

Probability of LS exceedance:

For $k^{th}$ limit-state,

$$P[LS\ exceedance] = P[C_k < EDP_k]$$

$$= \int_{\delta} P[C_k < EDP_k | EDP_k = \delta] \cdot f_{EDP_k}(\delta) \cdot d\delta$$

Fragility Function
# Experimental/Numerical Data Sources for Construction of Fragility Functions

The following table summarizes the sources of experimental and numerical data used for constructing fragility functions, including the specimen type, scale, and limit-state information:

<table>
<thead>
<tr>
<th>Sources</th>
<th>Specimen scale</th>
<th>Specimens</th>
<th>Limit-state</th>
</tr>
</thead>
<tbody>
<tr>
<td>Schoettler, Restrepo, Guerrini and Duck (2015)</td>
<td>full scale</td>
<td>1 single column bridge bent (dynamic test)</td>
<td>2</td>
</tr>
<tr>
<td>Barbosa, Link, and Trejo (2014)</td>
<td>half scale</td>
<td>6 column specimens with Grade 60 and Grade 80 steel</td>
<td>1</td>
</tr>
<tr>
<td>Goodnight, Kowalsky and Nau (2015)</td>
<td>half scale</td>
<td>23 column specimens of varying dimensions and reinforcement</td>
<td>1, 2</td>
</tr>
<tr>
<td>Murcia-Delso, Shing, Stavridis, and Liu (2013)</td>
<td>full scale</td>
<td>4 column specimens embedded in enlarged shafts</td>
<td>1, 2</td>
</tr>
<tr>
<td>Duck, Carreño, and Restrepo (2018)</td>
<td>FE model</td>
<td>36 numerical models of column reinforcement cages with varying parameters</td>
<td>3</td>
</tr>
<tr>
<td>Megally, Silva, and Seible (2002)</td>
<td>2/5&lt;sup&gt;th&lt;/sup&gt; scale</td>
<td>4 non-isolated exterior shear key specimens</td>
<td>4</td>
</tr>
<tr>
<td>Bozorgzadeh, Megally, Ashford, and Restrepo (2007)</td>
<td>2/5&lt;sup&gt;th&lt;/sup&gt; scale</td>
<td>1 isolated exterior shear key specimen</td>
<td>4</td>
</tr>
</tbody>
</table>
Limit-States: Limit State – 1 (Strain-based)

- Concrete cover crushing:
  Predictive Capacity Model:
  
  \[ EDP_{C_i}^{\text{PRED}} = \varepsilon_{\text{comp}}^{\text{bar}} = 0.00475 \]
  
  (Goodnight, Kowalsky and Nau, 2015)

![Experimental vs Predicted](image_url)

- Median = 1.02
- Dispersion = 32.6%

![Probability Plot](image_url)

Experimental value of strain at which concrete cover crushing is reached
Predicted value of strain at which concrete cover crushing is reached
Predictive Capacity Model:

\[ EDP_{C_2}^{\text{PRED}} = \varepsilon_{\text{tensile}}^{\text{bar}} = 0.03 + 700 \rho_s \frac{f'_{yhe}}{E_s} - 0.1 \frac{P}{f'_{ce}A_g} \]

(Goodnight, Kowalsky and Nau, 2015)

Longitudinal rebar buckling (a precursor):

Limit-States: Limit State – 2 (Strain-based)

Median = 1.05
Dispersion = 20.1%
Longitudinal rebar fracture (a precursor):

Predictive Capacity Model (mechanics-based):

\[
EDP^{\text{PRED}}_{C_2} = \max_t \varepsilon_{\text{bar}}^{\text{tensile}}(t) - \min_{t' > t} \varepsilon_{\text{comp}}^{\text{bar}}(t') = \\
0.11 + \min(0.054, 0.032, \rho_s \%) - 0.0175 \sqrt{n_{\text{bar}}} - 2.93 - 0.054 \frac{T}{\gamma} \\
\Delta\varepsilon_{V,K}
\]

\((\text{Duck, Carreño, and Restrepo, 2018})\)

\(\text{Median} = 0.99\)
\(\text{Dispersion} = 10.9\%\)

(Duck et al., 2018)
Parametric Probabilistic Seismic Performance Assessment

• Design variables & primary design parameter space
• Full-blown parametric risk-targeted seismic performance assessment and results
• Feasible design domains
**Primary design variables:**
1. Column diameter ($D_{col}$)
2. Column longitudinal reinforcement ratio ($\rho_{long}$)

subject to: \[ 1\% \leq \rho_{long} \leq 3\% \]

and

\[ D_{col} = 4 - 6 \text{ ft} \] for 4 column bents
\[ D_{col} = 5 - 8 \text{ ft} \] for 3 column bents
\[ D_{col} = 5 - 8 \text{ ft} \] for 2 column bents
\[ D_{col} = 5 - 8 \text{ ft} \] for 1 column bent

**Secondary design variables / components:**
1. Column transverse reinforcement ratio ($\rho_{trans}$)
2. Bridge deck
3. Bridge abutments
4. Foundations (piles and pile caps) to be capacity protected against other (undesirable) failure modes
Design variables: $d_i = [D_{col}, \rho_{long}]^T$

Probabilistic Seismic Hazard Analysis

Intensity Measure (IM):

$S_{x, avg} = \prod_{l=1}^{n} S_l (T_l)^{\frac{1}{2}}$

Attenuation Relationship:

$P[IM > x | L]$

Seismic Hazard Curve

$S_a$

VIM $10^0$

IM $10^0$

$T$

Averaging Period Range

Conditional Demand Model

$P[EDP > \delta | IM]$

Probabilistic Seismic Demand Hazard Analysis

Demand Hazard Curve

$V_{EDP}$

$10^0$

$10^3$

$EDP$

Probabilistic Seismic Damage Hazard Analysis

$\nu_{LS} = \frac{1}{\nu_{LS}} \int_{E_H} P[Z_i < 0 | EDP_j] dV_{EDP}$

Strain-based Fragility Functions

$P[Z_i < 0 | EDP_j]$

$C_i$: Capacity related to the $k^{th}$ limit-state

For $k^{th}$ limit-state, safety margin

$Z_i = C_i - EDP_i$
Results of Full-blown Parametric Risk-Targeted Seismic Performance Assessment

1. Concrete cover crushing
2. Longitudinal rebar buckling
3. Longitudinal rebar fracture
4. Shear key damage

$LS_1$ : Concrete cover crushing

$LS_2$ : Longitudinal rebar buckling

$LS_3$ : Longitudinal rebar fracture

$LS_4$ : Shear key damage
Results of Full-blown Parametric Risk-Targeted Seismic Performance Assessment: Feasible Design Domains
Development of Simplified Risk-targeted PBSD Method

- Obtaining a design point satisfying multiple risk-based objectives
- Approximation of feasible design domain
- Reduction in computational workload
Development of Simplified Risk-Targeted PBSD Procedure: Topology of Mean RP Surfaces

**LS₁ : Concrete cover crushing**

Target RP = 225 yr

**LS₂ : Longitudinal rebar buckling**

Target RP = 1000 yr

**LS₃ : Longitudinal rebar fracture**

Target RP = 2500 yr

\[ X [-] = \rho_{long} [-] + \frac{1}{m} [ft^{-1}] D_{col} [ft]; \ m = 0.005 \]
Development of Simplified Risk-Targeted PBSD Procedure: Finding a Design Point along a Positive Gradient Line

Equation of positive gradient line:

$$\rho_{\text{long}} [-] = m \left[ \text{ft}^{-1} \right] \cdot D_{\text{col}} \left[ \text{ft} \right] + \alpha [-]$$

As designed bridge

$L_{S_1}$ target contour (mean RP = 225 yr)

$L_{S_2}$ target contour (mean RP = 1000 yr)

$L_{S_3}$ target contour (mean RP = 2500 yr)

$$\rho_{\text{long}} = m \left( D_{\text{col}} \right) + \alpha$$

$$\left[\begin{array}{c}
\rho_{\text{long}} \\
\end{array}\right] = \left[\begin{array}{ccc}
m & \left( D_{\text{col}} \right) & \alpha \\
\end{array}\right]$$

$$\rho_{\text{long}} = \frac{1}{m} \left[ \text{ft}^{-1} \right] \cdot X \left[ \text{ft} \right] + \alpha \left[ \right]$$

$$X = \rho_{\text{long}} \left[ \right] + \frac{1}{m} \left[ \text{ft}^{-1} \right] \cdot D_{\text{col}} \left[ \text{ft} \right]$$

$$X^* = \max \left( (X^*)^{LS_1} , (X^*)^{LS_2} , \cdots , (X^*)^{LS_n} \right)$$
Development of Simplified Risk-Targeted PBSD Procedure: (Bi)Linear Approximation of Contour Lines

From full-blown parametric assessment

Consider $LS_2$
Development of Simplified Risk-Targeted PBSD Procedure: Approximate Feasible Design Domains

Bridge A

From full-blown parametric assessment

Bridge B

From full-blown parametric assessment

Bridge C

From full-blown parametric assessment

Bridge MAOC

From full-blown parametric assessment
Development of Simplified Risk-Targeted PBSD Procedure: Reduction in Computational Workload

Results obtained using 3 seismic hazard levels and 20 ground motions per hazard level.
Conclusions & Future Research Needs
Concluding Remarks

• Full-fledged probabilistic performance assessment of four Ordinary Standard Bridge (OSB) Testbeds in California using improved version of the PEER PBEE framework.
  ✓ Improved IM.
  ✓ Seismic hazard curve for improved IM.
  ✓ Conditional mean spectrum-based, site-specific, hazard/risk-consistent ground motion selection.
  ✓ Limit-states considered for RC bridge columns: (1) concrete cover crushing, (2) precursor to longitudinal rebar buckling, (3) a precursor to longitudinal rebar fracture.
  ✓ Material strain-based EDPs.
  ✓ Normalized strain-based fragility functions.

• Parametric full-fledged probabilistic performance assessment of four considered OSBs using a fully automated workflow.
  ✓ Investigate the effects of key structural design parameters on the mean RPs of limit-state exceedances.
  ✓ Topologies and contours of mean return period surfaces in the primary design parameter space.
  ✓ Target mean return periods of limit-state exceedances and feasible design domains.
  ✓ Full-fledged risk-targeted design framework.

• As-designed OSB testbed bridges considered exhibit significant variability in seismic performance as measured by the mean RPs of exceeding the selected set of limit-states.
Concluding Remarks & Future Research Needs

• Distilled out computationally more economical, simplified, non-traditional, risk-targeted PBSD method, building on the comprehensive probabilistic PEER PBEE framework, for Ordinary Standard Bridges (OSBs) in California.
  ✓ Find a design point in the primary design parameter space.
  ✓ Delineate approximate, sufficiently accurate, feasible design domain.

• Future Research Needs:
  ✓ Incorporation of (1) model parameter uncertainty, (2) parameter estimation uncertainty, and (3) modeling uncertainty.
  ✓ Explicit probabilistic treatment of near fault effects.
  ✓ Risk-targeted PBSD in terms of loss variables (e.g., life-cycle repair costs, downtime)
  ✓ Probabilistic explicit determination of secondary design variables to prevent undesirable failure modes with some specified level of confidence.
  ✓ Extend proposed simplified PBSD method to accommodate more than two primary design variables, especially for non-ordinary, more complex bridges.
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➢ Funding Support:
   California Department of Transportation (Caltrans)

➢ Technical Support and Insightful Discussions:
   Frank Beckwith (UC San Diego)
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   Peter Lee (Caltrans)
   Mark Mahan (Caltrans)
   Frank McKenna (UC Berkeley, NHERI SimCenter)
   Tom Ostrom (Caltrans)
   Tom Shantz (Caltrans)
   Charles Sikorsky (Caltrans)
Thank you!
Geometric mean of spectral accelerations at different periods (T₁, …, Tₙ):

\[
IM : \quad S_{a, \text{avg}}(T_1, \ldots, T_n) = \left[ \prod_{k=1}^{n} S_a(T_k) \right]^{1/n}
\]

Ref.: Kohrangi, Bazzurro and Vamvatsikos (2016)

\[
\nu_{S_{a, \text{avg}}}(s_a) = \sum_{i=1}^{N_{\text{faults}}} \nu_i \int_{R_i M_i} P\left[ \prod_{k=1}^{n} S_a(T_k) \right]^{1/n} > s_a \mid M_i = m, R_i = r \cdot f_{M_i}(m) \cdot f_{R_i}(r) \cdot dm \cdot dr
\]

\[
= \sum_{s=1}^{N_{\text{scenarios}}} P\left[ \prod_{k=1}^{n} S_a(T_k) \right]^{1/n} > s_a \mid \text{Scenario}_s \cdot \text{Rate(Scenario}_s) \]

Ref.: Kohrangi, Bazzurro and Vamvatsikos (2016)
Risk - Consistent Ground Motion Ensembles

Refs.: Baker and Jayaram (2011)
Kohrangi, Bazzurro, Vamvatsikos, and Spillatura (2017)
Previously Selected IM and Ground Motion Ensembles

Previously, ensembles of 40 ground motions were selected based on CMS \( (M = S_a(T_e = 1.0s)) \). \( T_e \) changed following a model update.

Note: Currently, ensembles of 40 ground motions were selected based on CMS \( (M = S_a(T_e = 1.0s)) \). \( T_e \) changed following a model update.
Probabilistic Seismic Demand Hazard Analysis

Seismic Hazard Curve

no. of Hazard Levels

\[ V_{S_{a, \text{avg}}} \quad S_{u, \text{avg}} \quad 10^0 \]

\[ S_d \]

\[ EDP_2: \max \left( \max_t \left( \bar{\varepsilon}_{\text{tensile}} \right) \right) \]

\[ IM : S_{u, \text{avg}} [g] \]

\[ \eta_{EDP_2,IM} \]

Lognormal \( f_{EDP_2,IM} \) (edp2/IM)

Regressed Median, 16th, and 84th percentile EDP2/IM

Regressed Lognormal \( f_{EDP_2,IM} \) (edp2/IM)

\[ \lambda_{EDP_2,IM} \]

\[ \sigma_{EDP_2,IM} \]

\[ \nu_{EDP_2,IM} \]

\[ \lambda_{EDP_2,IM} \]

\[ \nu_{EDP_2,IM} \]
\[ \nu_{EDP_k}(\delta) = \int_{IM} P[EDP_k > \delta \mid IM = x] \cdot d\nu_{IM}(x) \]

\[ \nu_{IM}(x + dx) - \nu_{IM}(x) \]

\[ \nu_{IM}(x) \]

**Probabilistic Seismic Demand Hazard Analysis**

**Demand Hazard Curve**

**IM Deaggregation of Demand Hazard**

**Regressed Median, 16th, and 84th percentile EDP2|IM**

**Regressed Lognormal \( f_{EDP2|IM}(edp2|IM) \)**

**\( EDP_2 : \max_{\text{column}} \left( \max_{\text{bar}} \left( \max_{t} \varepsilon_{\text{tensile}}(t) \right) \right) \)**

\[ IM : S_{a, \text{avg}} [g] \]

\[ RP = 72 \text{ yrs} \]

\[ RP = 975 \text{ yrs} \]

\[ RP = 4975 \text{ yrs} \]
Probability density function of $EDP_k$:

$$f_{EDP_k}(\delta) = \frac{d}{d\delta} \left( 1 - \frac{\nu_{EDP_k}(\delta)}{\nu_{IM}(x = 0)} \right)$$

Probability of LS exceedance:

For $k^{th}$ limit-state,

$$P[LS \text{ exceedance}] = P[C_k < EDP_k]$$

$$= \int_{\delta} P[C_k < EDP_k | EDP_k = \delta] \cdot f_{EDP_k}(\delta) \cdot d\delta$$

Fragility Function
Limit-States: Limit State – 4 (Displacement-based)

- Exterior shear key reaching its shear strength capacity

Predictive Capacity Model:

\[ EDIP_{C4}^{\text{PRED}} = \Delta_{C}^{SK} = \sqrt{2} \varepsilon_y (L_d + b) \frac{h + d_1}{s} \]

(Megally et al., 2002)

Non-isolated shear key Isolated shear key

(Megally et al., 2002) (Bozorgzadeh et al., 2007)

Median = 1.14 (non-isolated), 1.0 (isolated)
Dispersion = 11.06% (non-isolated), 0.0% (isolated)
Concluding Remarks

• Assessment of four Ordinary Standard Bridge (OSB) Testbeds in California using improved version of the PEER PBEE framework.
  ✔ Use of an improved IM consisting of the average spectral acceleration over a specified period range.
  ✔ Derivation of seismic hazard curve for improved IM in terms of the results of standard PSHA for spectral accelerations at single periods.
  ✔ Conditional mean spectrum-based, site-specific, hazard/risk-consistent ground motion selection.
  ✔ Limit-states considered for RC bridge columns: (1) concrete cover crushing, (2) a precursor to longitudinal rebar buckling, (3) a precursor to longitudinal rebar fracture.
  ✔ Material strain-based EDPs associated with limit-states considered.
  ✔ Normalized strain-based fragility functions based on reliable experimental data or high-fidelity numerical data.

• Parametric full-fledged probabilistic performance assessment of four considered OSBs using a fully automated workflow in parallel computing environment.
  ✔ Investigate the effects of key structural design parameters parameters on the mean RPs of limit-state exceedances.
  ✔ Topologies and contours of mean return period surfaces in the primary design parameter space.
  ✔ Target mean return periods of limit-state exceedances and feasible design domains.
Concluding Remarks

• Probabilistic PBSD for California Ordinary Bridges with performance objectives explicitly stated in terms of the risk associated with the exceedance of critical damage/limit states
  ✓ Provides an
• Distilled out a computationally more economical, simplified, non-traditional, risk-targeted PBSD method, building on the comprehensive probabilistic PEER PBEE framework, for Ordinary Standard Bridges (OSBs) in California.
  ✓ Find a design point in the primary design parameter space.
  ✓ Delineate approximate, sufficiently accurate, feasible design domain.
• Seismic performance of the as-designed OSB testbed bridges considered shows significant variability of seismic performance as measured by the mean RPs of exceeding the selected set of limit-states.
  ✓ Limit-state 1: mean RP = 150 – 1,500 years
  ✓ Limit-state 2: mean RP = 500 – 10,000 years
  ✓ Limit-state 3: mean RP = 1,000 – 30,000 years
  ✓ Limit-state 4 (abutment exterior shear key reaching its shear strength capacity): 80 – 2,500 years
Concluding Remarks

• Future research needs:
  ✓ Incorporation of (1) model parameter uncertainty, (2) parameter estimation uncertainty, and (3) modeling uncertainty.
  ✓ Explicit probabilistic treatment of near fault effects.
  ✓ Risk-targeted PBSD in terms of loss variables (life-cycle repair costs, downtime)
  ✓ Develop probabilistically explicit determination of secondary design variables to prevent undesirable failure modes with some specified level of confidence.
  ✓ Extend proposed simplified PBSD method to accommodate more than two primary design variables, especially for non-ordinary bridges.