Multi-Fidelity Modelling for Collapse Simulation of Steel and Composite Structures with OpenSees

WINTER SCHOOL
5th International course on
Seismic Analysis of Structures using OpenSEES
Finite Element-based Framework and Civil Engineering Applications

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Politecnico di Torino
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Acknowledgements - Research Group

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Acknowledgements
Large Earthquakes In Recent Years

- Demonstrated that the magnitude of earthquake forces applied to buildings were significantly larger than the corresponding design earthquake loads based on regional seismic design.

Images Source: EERI Reconnaissance Reports
Collapse Risk During Large Earthquakes

- Hyogoken-Nanbu 1995
- Taiwan 1999
- Loma Prieta 1989
- Northridge 1994
- Hyogoken-Nanbu 1995
- Tohoku 2011

Images Source: NISEE, E-Library

-we need to know the ability of structures to withstand a given level of demand without suffering severe deterioration right after an extreme seismic event.
### Need for Collapse Simulations
- (Within a Design Code) Building Seismic Performance Factors
- System Overstrength, Strength Reduction Factors

<table>
<thead>
<tr>
<th>Structural Type</th>
<th>DCM $q$</th>
<th>DCH $q$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment-Resisting Frames</td>
<td>4</td>
<td>$5\alpha_u / \alpha_1$</td>
</tr>
<tr>
<td>Concentrically Braced Frames</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>• Diagonal Bracing</td>
<td>2</td>
<td>2.5</td>
</tr>
<tr>
<td>• V-Bracing</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Eccentrically Braced Frame</td>
<td>4</td>
<td>$5\alpha_u / \alpha_1$</td>
</tr>
</tbody>
</table>

“Normally”, conduct collapse simulations other than just nonlinear static analysis to get those factors for new systems

FEMA P695
Need for Nonlinear Analysis with Emphasis at Collapse
- Seismic Evaluation and Retrofit of Existing Structures

Seismic Evaluation and Retrofit of Existing Buildings

This document uses both the International System of Units (SI) and customary units

Guevara-Perez (2012)
Need for Collapse Simulations
- Performance-Based Earthquake Engineering
- Mathematical Formulation (Cornell-Krawinkler Equation)

\[ \lambda(DV) = \int \int \int G(DV \mid DM) dG(DM \mid EDP) dG(EDP \mid IM) d\lambda(IM) \]
What is Important for Tracing Earthquake-induced Collapse?

9-story Frame, $T_1 = 1.80$sec, Motion: NR94nya

- Non-degrading system
- Degrading system

P-Delta Only

Collapse Capacity

Image Credit: Prof. Helmut Krawinkler
Models that Facilitate Nonlinear Simulations

✧ Limitations apply depending on the modeling option
✧ Trade off between simplicity and computations
✧ The model (and level of sophistication) should be conditioned to the performance objective
Lateral Load Resisting Systems of Interest

Steel Moment-Resisting Frames

Steel Concentrically Braced Frames

Images courtesy of Prof. M. Engelhardt
Typical Modeling Approach within One Frame Bay

Image courtesy of Prof. M. Engelhardt

Image: Akcelyan and Lignos (2017)
Nonlinear Modeling Approach

- Steel Beams
- Steel Columns
- Steel braces

Image Source: NIST GSR 10-917-5
Nonlinear Modeling Guidelines for Steel Beams

Fully-Restrained Bare and Composite Beam-to-Column Connections

Image Source: Lignos et al. (2013)
Steel Beam Ductility Under Earthquake Loading

- Local buckling
- Lateral torsional buckling

D.G. Lignos: Multi-fidelity modeling for collapse simulation of steel and composite structures
Steel Beams - Concentrated Plasticity Deterioration Models
The Modified Ibarra-Medina-Krawinkler (IMK) Deterioration Model

Reference Energy Dissipation Capacity

\[
E_t = \lambda \cdot \theta_p \cdot M_y \quad M_i = (1 - \beta_i) \cdot M_{i-1} \quad K_i = (1 - \beta_i) \cdot K_{i-1} \quad \beta_i = \left( \frac{E_i}{E_t - \sum_{i=1}^{i-1} E_j} \right)^c
\]
D.G. Lignos: Multi-fidelity modeling for collapse simulation of steel and composite structures

modified IMK Model in OpenSees


Modified Barra Krawinkler Deterioration Model

- Image of the modified IMK model diagram.
Utilizing the Modified IMK Model in OpenSees - Sample Model Calibrations

Steel Beam (bare) with RBS

Steel Beam (Composite) with RBS

(Source: Lignos and Krawinkler 2011)

(Source: Elkady and Lignos 2014)

Utilizing the Modified IMK Model in OpenSees

Steel Beams (bare) with RBS

\[ M^*_y = 1.1M_{pe} = 1.1 \cdot R_y \cdot Z \cdot F_y = 1.1 \cdot R_y \cdot W_{ply} \cdot F_y \]

\[ M_u = 1.1M^*_y \ (COV=0.1) \]

\[ M_r = 0.4M^*_y \]

Plastic Deformation Parameters

\[ \theta_p = 0.19 \left( \frac{h}{t_w} \right)^{-0.314} \left( \frac{b_f}{2 \cdot t_f} \right)^{-0.10} \left( \frac{L_b}{r_y} \right)^{-0.185} \left( \frac{L}{d} \right)^{0.113} \left( \frac{c_{unit} \cdot d}{533} \right)^{-0.76} \left( \frac{c_{unit} \cdot F_y}{355} \right)^{-0.07} \quad COV = 0.24 \]

\[ \theta_{pc} = 9.52 \left( \frac{h}{t_w} \right)^{-0.513} \left( \frac{b_f}{2 \cdot t_f} \right)^{-0.863} \left( \frac{L_b}{r_y} \right)^{-0.108} \left( \frac{c_{unit} \cdot F_y}{355} \right)^{-0.36} \quad COV = 0.26 \]

\[ \theta_u = 0.20 \text{rad} \]

Reference Energy Dissipation Capacity

\[ \Lambda = 585 \cdot \left( \frac{h}{t_w} \right)^{-1.14} \left( \frac{b_f}{2 \cdot t_f} \right)^{-0.632} \left( \frac{L_b}{r_y} \right)^{-0.205} \left( \frac{c_{unit} \cdot F_y}{355} \right)^{-0.391} \quad COV = 0.35 \]

(Source: Lignos and Krawinkler 2011)

Databases for Calibration of Deterioration Models
-Databases publicly available from: resslabtools.epfl.ch
Composite Steel Beams

Bare Beam with RBS
(Symmetric Hysteretic Response)

Composite Beam with RBS
(Assymmetric Hysteretic Response)

Data from Uang et al. (2000)

Data from Ricles et al. (2004)
Modeling Guidelines for Composite Beams

(Elkady and Lignos 2014; El-Jisr et al. 2019)


Modeling of Steel Braces under Cyclic Loading
-Modern Steel Braced Frames

\[ N/N_{pl} \]

\[ \frac{N}{N_{max}} \]

\[ A_g F_y \]

\[ \mu \]

\[ N_{b1}, N_{b2}, N_{b3} \]

\[ N_{b, i} \]: Steel brace buckling resistance at ductility level, \( i \)

\[ \mu \]: Ductility range
Gusset Plate flexibility and yield moment are modeled according to the model proposed by Roeder et al. (2011).

Coffin-Manson fracture initiation criterion \[ \varepsilon_i = \varepsilon_o \left( \frac{N_f}{N_y} \right)^m \]

EPFL Steel Brace Databases

SOON PUBLICALLY AVAILABLE FROM: resslabtools.epfl.ch

✧ Collected Data from 24 different experimental programs since 1970s, organized in a consistent format (Metadata + Results)

Examples of typical configurations (Recent system level studies are also included)

→ Fully digitized data of axial load axial displacement relationships

(Karamanci et al. 2012*)

Illustrative Calibration Examples

(Data from Fell et al. 2009)

(Data from Wakabayashi et al. 1977)

(Karamanci and Lignos 2014*)

Proposed Equation:

\[ \varepsilon_0 = 0.291 \left( \frac{kL}{r} \right)^{-0.484} \left( \frac{w}{t} \right)^{-0.613} \left( \frac{E}{F_y} \right)^{0.3} \]

Range of Applicability for Predictors:

\[ 27 \leq \frac{kL}{r} \leq 85 \]
\[ 4.2 \leq \frac{w}{t} \leq 30.40 \]
\[ 223 \leq F_y \leq 532 \text{ MPa} \]

<table>
<thead>
<tr>
<th>Brace Component</th>
<th>Section</th>
<th>Steel Material (Menegotto-Pinto)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Segments</td>
<td>Integration points</td>
<td># Fibers (w)</td>
</tr>
<tr>
<td>8</td>
<td>5</td>
<td>10</td>
</tr>
</tbody>
</table>
Proposed Equation:

\[ \varepsilon_0 = 0.748 \left( \frac{kL}{r} \right)^{-0.399} \left( \frac{D}{t} \right)^{-0.628} \left( \frac{E}{F_y} \right)^{0.2} \]

Range of Applicability for Predictors:

\[ 29 \leq \frac{kL}{r} \leq 128 \]
\[ 12.75 \leq \frac{D}{t} \leq 39.91 \]
\[ 326 \leq F_y \leq 521 MPa \]

(Karamanci and Lignos 2014*)

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<tbody>
<tr>
<td>Segments</td>
<td>Integration points</td>
<td># Fibers (D)</td>
</tr>
<tr>
<td>8</td>
<td>5</td>
<td>12</td>
</tr>
</tbody>
</table>
**Proposed Equation:**

\[ \varepsilon_0 = 0.0391 \left( \frac{kL}{r} \right)^{-0.234} \left( \frac{b_f}{2t_f} \right)^{-0.169} \left( \frac{T}{t_w} \right)^{-0.065} \left( \frac{E}{F_y} \right)^{0.351} \]

**Range of Applicability for Predictors:**

\( 39 \leq \frac{kL}{r} \leq 153 \)

\( 4.19 \leq \frac{b_f}{2t_f} \leq 10.20 \)

\( 7.99 \leq \frac{T}{t_w} \leq 49.40 \)

\( 284 \leq F_y \leq 414\text{MPa} \)

**Brace Component** | **Section** | **Steel Material (Menegotto-Pinto)**
---|---|---
Segments | Integration points | \# Fibers \((T, b_f)\) | \# Fibers \((t_f, t_w)\) | \(b\) | \(R_o\) | \(a_1\) | \(a_3\) | \(m\)
8 | 5 | 12 | 4 | 0.1% | 20 | 0.01 | 0.02 | -0.3

(Karamanci and Lignos 2014*)
Nonlinear Modeling Guidelines for Steel Columns

Image from Elkady and Lignos (2017)

D.G. Lignos: Multi-fidelity modeling for collapse simulation of steel and composite structures
Steel Column (I-Shape) Database

SOON PUBLICALLY AVAILABLE FROM: resslabtools.epfl.ch

Source: Elkady and Lignos (2018*)

Steel Column (I-Shape) Database
-Calibration Examples for Steel I-shape Columns

Source: Lignos et al. (2019*)

Modeling Guidelines for the Modified IMK Model
Wide-Flange (I-Shape) Steel Columns

If \( P_g / P_{ye} \leq 0.20 \), \( M_y^* = 1.15 \cdot Z \cdot R_y \cdot F_y \left(1 - P_g / P_{ye}\right), (COV = 0.10) \)

If \( P_g / P_{ye} > 0.20 \), \( M_y^* = 1.15 \cdot Z \cdot R_y \cdot F_y \left[\frac{9}{8} \left(1 - P_g / P_{ye}\right)\right], (COV = 0.10) \)

\[ M_u = a \cdot M_y^* \quad a = 12.5 \left(\frac{h}{t_w}\right)^{-0.2} \left(\frac{L}{r_y}\right)^{-0.4} \left(1 - \frac{P_g}{P_{ye}}\right)^{0.4} \geq 1.0, \ & a \leq 1.3, \ (COV = 0.10) \]

\[ M_r = \left(0.5 - 0.4 \frac{P_g}{P_{ye}}\right) M_y^* \ (COV = 0.27) \]

\[ \theta_p = 294 \left(\frac{h}{t_w}\right)^{-1.7} \left(\frac{L}{r_y}\right)^{-0.7} \left(1 - \frac{P_g}{P_{ye}}\right)^{1.6} \leq 0.20 (COV = 0.39) \]

\[ \theta_{pc} = 90 \left(\frac{h}{t_w}\right)^{-0.8} \left(\frac{L}{r_y}\right)^{-0.8} \left(1 - \frac{P_g}{P_{ye}}\right)^{2.5} \leq 0.30, (COV = 0.14) \]

\[ \theta_u = 0.15, (COV = 0.46) \]

Range of applicability

\[ 3.71 \leq h / t_w \leq 57.5 \]
\[ 1.82 \leq b_f / 2t_f \leq 8.52 \]
\[ 38.4 \leq L / r_y \leq 120 \]
\[ 0 \leq P_g / P_{ye} \leq 0.75 \]

Lignos et al. (2019*)

If \( P_g / P_{ye} \leq 0.20 \), \( M_y^* = 1.15 \cdot Z \cdot R_y \cdot F_y \left( 1 - P_g / P_{ye} \right), (COV = 0.10) \)

If \( P_g / P_{ye} > 0.20 \), \( M_y^* = 1.15 \cdot Z \cdot R_y \cdot F_y \left[ \frac{9}{8} \left( 1 - P_g / P_{ye} \right) \right], (COV = 0.10) \)

\[ M_u = a \cdot M_y^* \quad a = 0.04 \left( \frac{D}{t} \right)^{-0.3} \left( 1 - \frac{P_g}{P_{ye}} \right)^{1.3} \left( \frac{E}{F_{ye}} \right)^{-0.75} \geq 1.0, \text{ and } a \leq 1.3, \text{ (COV = 0.23)} \]

\[ M_r = \left( 0.5 - 0.6 \frac{P_g}{P_{ye}} \right) M_y^* \text{ (COV = 0.34)} \]

\[ \theta_p = 0.3 \left( \frac{D}{t} \right)^{-0.95} \left( 1 - \frac{P_g}{P_{ye}} \right)^{1.1} \left( \frac{E}{F_{ye}} \right)^{0.1} \text{ (COV = 0.26)} \]

\[ \theta_{pc} = 5.4 \left( \frac{D}{t} \right)^{-1.2} \left( 1 - \frac{P_g}{P_{ye}} \right)^{3.0} \left( \frac{E}{F_{ye}} \right)^{0.14} \text{ (COV = 0.35)} \]

\[ \theta_u = 0.10, \text{ (COV = 0.50)} \]

**Range of applicability**

\[ 20 \leq D / t \leq 40 \]

\[ 0 \leq P_g / P_{ye} \leq 0.60 \]

\[ 40 \leq F_{ye} \leq 72.5 \text{ksi} \]

Limitations of Concentrated Plasticity Models
-Column Axial Shortening

Source: Suzuki and Lignos (2015)

New Fiber-Based Deterioration Model

Material subclass

... new material

- Force-based formulation (1 element)
- Seven integration points
- Midpoint integration
- HSS-Shapes: 2x8 fibers (Flat) 2x4 (Corner)
- W-Shapes: 2x8 fibers (Flg.) 2x8 fibers (Web)

Source: Suzuki and Lignos (2017)

New Fiber-Based Deterioration Model

✧ Monotonic Stress-Strain Response

Source: Suzuki and Lignos (2017)

New Fiber-Based Deterioration Model

 الحالي

Cyclic Stress-Strain Response (Pre-Buckling)

New Fiber-Based Deterioration Model

✧ Cyclic Stress-Strain Response (Post-Buckling)

Axial load

![Image of steel structure]

Inelastic buckling: controlled by cumulative plastic strain

Source: Suzuki and Lignos (2017)

Proposed Deterioration Model for Steel Columns
- Validation with Experimental Data: Symmetric Loading History

Source: Suzuki and Lignos (2017)

Proposed Deterioration Model for Steel Columns

- Validation with Experimental Data: Symmetric Loading History

Source: Suzuki and Lignos (2017)

Geometric Instabilities We Cannot Trace with Existing Modeling Capabilities in OpenSees

Out-of-plane instabilities

Global and local Imperfections

Source: Elkady and Lignos (2018)
System Level Validations
Simulating Structural Collapse in OpenSees
- Illustration: 8-Story Building (Video from Elkady and Lignos 2015)
Earthquake-induced Collapse Assessment
-Case Study 1: Steel Moment Resisting Frames (MRFs)

- Design Codes: IBC-2003, AISC-2005
- Beams with RBS
- Design Area: Los Angeles
- $T_1=1.32\text{sec}$

Source: Lignos et al. (2011)*

Scale Models for Shaking Table Collapse Tests - Case Study 1: Steel (MRFs)

1/8 Scale model

Typical plastic hinge location of Test Model

Source: Lignos et al. (2011)*
Scale Models for Shaking Table Collapse Tests
-Case Study 1: Steel (MRFs)

Source: Lignos et al. (2011)*
Typical Connection Hysteretic Behavior
-Case Study 1: Steel (MRFs)

Source: Lignos et al. (2011)*
Ground Motion for Collapse Tests
-Case Study 1: Steel MRFs

Northridge 1994 Canoga Park: Acceleration Spectrum, $\zeta=5\%$
Collapse Tests: NEES @ Buffalo NY

Source: Lignos et al. (2011)*
TESTING PROGRAM AND ASSESSMENT OF RESULTS NEAR COLLAPSE

Each frame was subjected to four series of tests on the earthquake simulator as summarized in Table 1 together with the testing phase notation. The last series (CLEF: “Final” Collapse Level Earthquake) was conducted because both frames did not collapse as expected during the Collapse Level Earthquake (CLE) series as suggested by pre-test predictions. Since the focus is on behavior near collapse, the emphasis in the following discussion is placed on the last two series (CLE and CLEF). A detailed assessment of all tests is presented in Lignos and Krawinkler, (2009). Frame #1 was subjected to the Northridge 1994 Canoga Park record with accelerations scaled incrementally. Figure 3a shows the roof drift history of Frame #1 together with analytical predictions based on the post–test numerical model discussed in Lignos and Krawinkler, (2009).

After drifting in one direction (ratcheting) Frame #1 collapsed during the first cycle of the CLEF with a complete 3-story collapse mechanism (see Figure 3a). For Frame #2 during the Maximum Considered Earthquake (MCE) the 150% Chile 1985 Llolleo record was used since the intent was to investigate the effect of cumulative damage on the collapse capacity of the test frame. Due to unsuccessful reproduction of the input ground motion residual deformations of Frame #2 after MCE were the same as after DLE. During CLEF Frame #2 finally collapsed in the opposite direction compared to Frame #1 but with the same collapse mechanism. Numerical simulations of roof drift histories for CLE and CLEF phase of Frame #2 are presented in Figure 3b and are in a good agreement with experimental results.

Source: Lignos et al. (2011)*
System Level Collapse Simulation Validations
Case Study 2: E-Defense Full-Scale Collapse Tests

(Objective:
- Numerical prediction of collapse
- Ways to mitigate (delay) collapse

Base Column Local Buckling

(Photo Source: E-Defense-2007, Suita et al. 2008)
System Level Collapse Simulation Validations
Case Study 2: E-Defense Full-Scale Collapse Tests

D.G. Lignos: Multi-fidelity modeling for collapse simulation of steel and composite structures
Composite Steel Beam Modeling
Case Study 2: E-Defense Full-Scale Collapse Tests

Moment [kN·m] vs. Rotation θ [rad]

Exper. Data
Simul. Data

(Data from Pre-Test E-Defense Blind Competition)
Composite Beam under cyclic loading

Source: Lignos et al. (2013)*

Steel Column Model (HSS Modeling)

- ALC Panels
- Safeguard System

Steel Column Model (HSS Modeling)

Rigid bar element
Panel spring: Kwon-Harker non-linear model
Column hinge: modified IMK deterioration model
Beam hinge: Modified IMK deterioration model
Elastic element
Final base

Moment M
Rotation θ

Stress
Strain

Engineering stress-strain

Source: Suzuki and Lignos (2019)*

D.G. Lignos: Multi-fidelity modeling for collapse simulation of steel and composite structures
Validation with Full-Scale Collapse Tests

Image Source: Lignos et al. (2013)*


Looking into Local Engineering Demand Parameters (EDPs)

Axial Shortening [mm]

Column Rotation [rad]

Column 1 (end)  Column 2 (interior)  Column 3 (end)

Differential gap

Local buckling

Source: Suzuki and Lignos (2019)*

1-Story Concentrically Braced Frame
-Case Study 3: Shake Table Test Validation

Source: Okazaki et al. 2013*

1-Story Concentrically Braced Frame
-Case Study 3: Test Frame

Source: Okazaki et al. 2013*

1-Story Concentrically Braced Frame
-Case Study 3: Model Description

1-Story Concentrically Braced Frame
-Case Study 3: Shake Table Test Validation

Source: Okazaki et al. 2013*

1-Story Concentrically Braced Frame
-Brace Response and Predictions

East Brace

Source: Okazaki et al. 2013*

1-Story Concentrically Braced Frame
-Base Base and Simulations

Source: Okazaki et al. 2013*
Few Words for the Future

Out-of-plane instabilities

Severe panel zone shear distortion
(existing steel buildings)

Source: Elkady and Lignos (2017)
New Multiaxial Plasticity Model for Steels

Updated Voce-Chaboche Model for Mild Steels

\[
\sigma_y = \sigma_{y,0} + Q_\infty (1 - \exp[-b \varepsilon_{eq}^p]) - D_\infty (1 - \exp[-a \varepsilon_{eq}^p])
\]

Source: Hartloper et al. (2019)*

UVCuniaxial (Updated Voce-Chaboche)

Command Manual

This command is used to construct an Updated Voce-Chaboche (UVC) material for uniaxial stress states (e.g., beam elements). This material is a refined version of the classic nonlinear isotropic/kinematic hardening material model based on the Voce isotropic hardening law and the Chaboche kinematic hardening law. The UVC model contains an updated isotropic hardening law, with parameter constraints, to simulate the permanent decrease in yield stress with initial plastic loading associated with the discontinuous yielding phenomenon in mild steels.

Details regarding the model, its implementation, and calibrations can be found in the references cited at the end. The multiaxial9 (e.g., for solid/brick elements) and plane-stress9 (e.g., for quadrilateral/shell elements) versions are also available. The multiaxial and plane-stress implementations have the exact same hardening rules as this uniaxial model, and only differ in their purpose and numerical implementation.

Available in OpenSees version 3.14.0.

UniaxialMaterial UVCuniaxial SmartTag 1E by 50DF 5E 50DF 6E SM SC1 $gamma1 sc2 $gamma2 sc3 $gamma3 sc8 $gamma8

\( \text{Inputs:} \) UVCuniaxial SmartTag 1E by 50DF 5E 50DF 6E SM SC1 $gamma1 sc2 $gamma2 sc3 $gamma3 sc8 $gamma8

\( \text{Outputs:} \) UVC uniaxial

Source: Hartloper et al. (2019)*

Optimization Framework for Consistent Material Parameters for UVCuniaxial

✧ Objective function

\[ \varphi = \sum_{Tests} \frac{\int_0^{\varepsilon^*} (\sigma_{model} - \sigma_{Test})^2 d\varepsilon^*}{\int_0^{\varepsilon^*} d\varepsilon^*} \]

✧ Optimization Solution

✧ Newton Trust-Region Method (NTR)
  ✧ Jacobi preconditioning
  ✧ Singular Value Decomposition (SVD) preconditioning

Source: de Castro e Sousa A., et al. (2019)*

Optimization Framework for Consistent Material Parameters

Source: de Castro e Sousa A., et al. (2019)*

RESSLab Python (RESSPyLab)

Welcome to the Resilient Steel Structures Laboratory (RESSLab) Python Library.

The RESSLab is a research laboratory at École Polytechnique Fédérale de Lausanne (EPFL).

The library is currently under testing phase.

- Free software: MIT license
- Documentation: https://RESSPyLab.readthedocs.io.

Features
- Implicit integration scheme for non-linear uniaxial Voce and Chaboche metal plasticity
- Newton Trust-Region (NTR) with Singular Value Decomposition (SVD) and Jacobi(J) preconditioning solver
- Voce and Chaboche material parameter estimation with NTR-SVD and NTR-J

Credits
**Proposed Library of Material Model Parameters for Different Steels Worldwide**

<table>
<thead>
<tr>
<th>Material</th>
<th>E</th>
<th>fy</th>
<th>Qinf</th>
<th>b</th>
<th>Dinf</th>
<th>a</th>
<th>C1</th>
<th>gamma1</th>
<th>C2</th>
<th>gamma2</th>
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<td>72</td>
<td>73</td>
<td></td>
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<tr>
<td>S355J2+N (25 mm plate)</td>
<td>197.41</td>
<td>338.80</td>
<td>134.34</td>
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<td>S355J2+N (HEB500 web)</td>
<td>199.68</td>
<td>334.94</td>
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<td>187.61</td>
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<td>97.35</td>
<td>14.02</td>
<td>136.64</td>
<td>226.40</td>
<td>26691.00</td>
<td>188.75</td>
<td>2892.40</td>
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<tr>
<td>S690QL (25 mm plate)</td>
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<td>0.11</td>
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<td>285.15</td>
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<td>185.16</td>
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<tr>
<td>A992 Gr. 50 (W14X82 web)</td>
<td>210.74</td>
<td>378.83</td>
<td>122.63</td>
<td>19.74</td>
<td>143.49</td>
<td>248.14</td>
<td>31638.00</td>
<td>277.32</td>
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<td>105.95</td>
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<td>200.43</td>
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<td>0.09</td>
<td>103.30</td>
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<td>225.26</td>
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<td>HYP400 (27mm plate)</td>
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</table>

*Source: Hartloper et al. (2019)*

High-Fidelity Panel Zone Modeling in OpenSees

Source: Skiadopoulos and Lignos (2020)*
In a conventional PC, the OpenSees model is 60 to 80 times faster than the ABAQUS model.

Source: Skiadopoulos and Lignos (2020)*
Comparisons with Existing Modeling Approaches

Source: Skiadopoulos and Lignos (2020)*
Concluding Remarks

✧ Data-driven framework for multi-fidelity nonlinear modeling of steel and composite members
✧ Explicit guidelines for nonlinear modeling:
  ✧ Steel beams (bare/composite)
  ✧ Steel columns
  ✧ Steel braces
✧ Validated with system-level collapse experiments
✧ Model fidelity should be benchmarked with the performance-objective and EDP of interest (global or local)
✧ There is still much work that needs to be done
Relevant Publications


Relevant Publications (2)


Thank you for your kind attention!

For more information visit: resslab.epfl.ch