Risk analysis of transmission lines systems to natural hazards with emphasis on earthquakes and extreme winds

Seismic Collapse Risk of a lattice Transmission Tower

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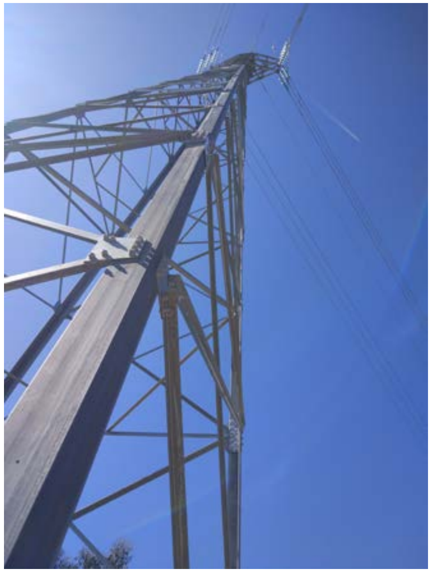


Outline

- Overhead Line System – Quick Review of the **Case study**;
  - Overhead Power Line Layout;
  - Lattice tower description;

- **Capacity assessment of a lattice transmission tower**
  - Numerical modeling in OpenSees of a Lattice Tower (Update)
  - Hysteretic behavior of a single angle member and a cross-brace panel
  - Pushover analysis under different loading patterns

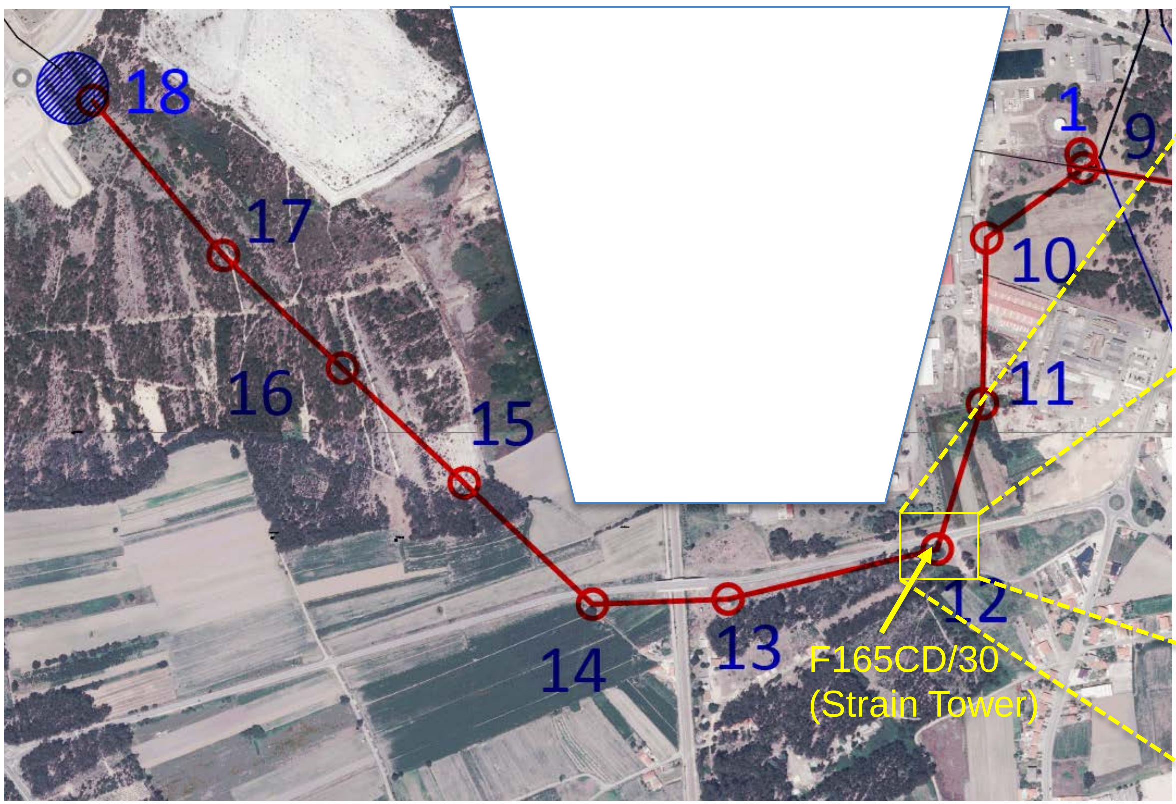
- **Seismic Collapse Risk assessment of a lattice Transmission Tower**
  - Assessment of the annual collapse rate as a risk metric
  - Collapse data obtained through an Incremental Dynamic Analysis;

- Conclusions and future developments

Case Study Presentation – Line Layout

Overhead High Voltage Line - Sub-transmission **60 kV Line** from EDP,DISTRIBUIÇÃO located in the North of Portugal

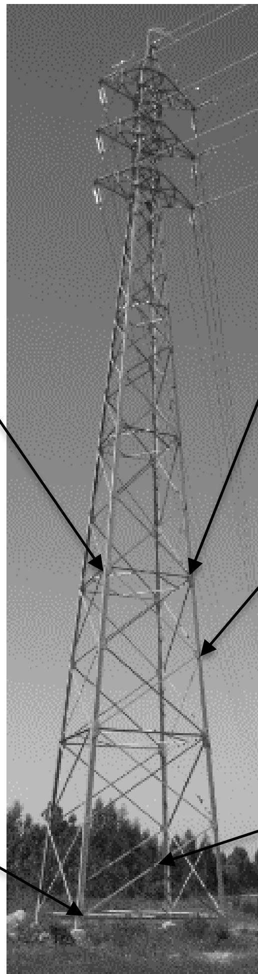
Overhead Power-Line Layout



Previously RADAR Monitored Tower

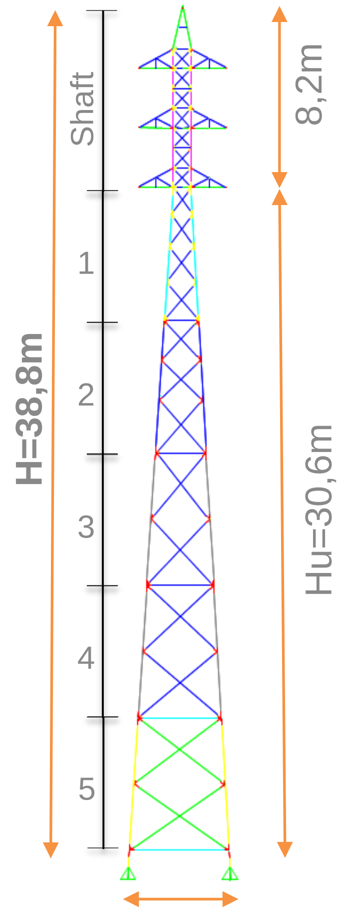
Case Study Presentation – 3D View

Overhead High Voltage Line - Sub-transmission **60 kV Line** from EDP,DISTRIBUIÇÃO located in the North of Portugal



Brace connection

Pinned joint in brace intersection



B=5,172 m (square base)

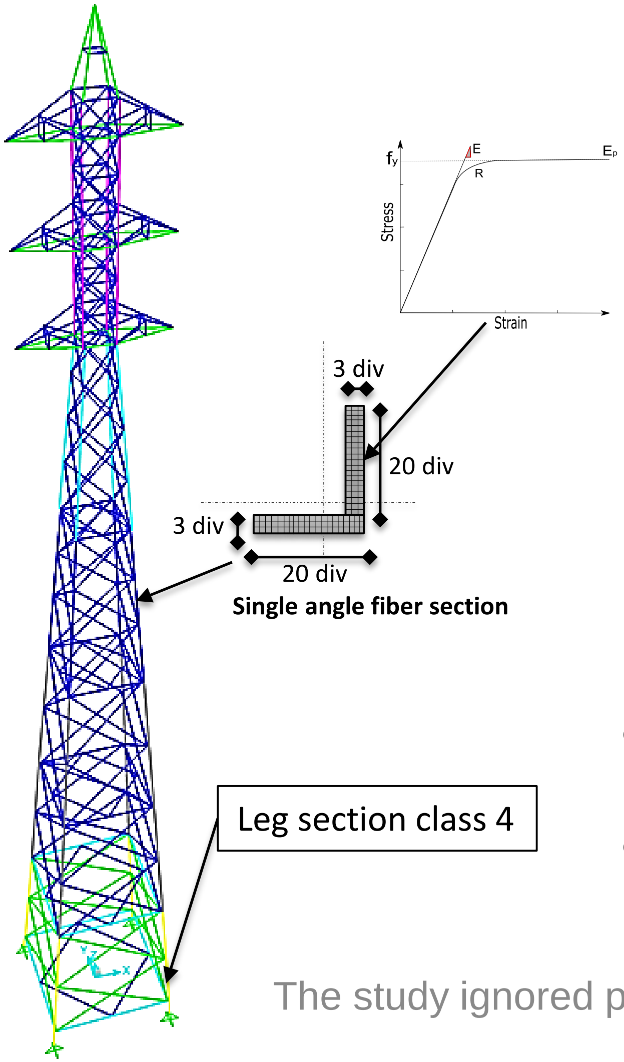
F165CD/30

Tower Member	Sections (angle)
Leg	L180x16 (yellow) L180x15 (grey) L160x15 (blue) L140x13 (cyan) L100x10 (magenta)
Diagonal Bracing	L60x6 (green) L50x5 (blue)
Horizontal Bracing	L70x7 (cyan) L50x5 (blue)
Cross arm (L=1,8 m)	L50x5 (blue) sup. chord L60x6 (green) inf. chord

Steel class - S275

Capacity assessment of a lattice transmission tower

Numerical modeling in OpenSees of a Lattice Tower



- Forced-based elements (FBE) were used for the main members
  - Fiber section were considered to generate the cross section)
  - Three integration points per element
  - **Eight FBE** per member to capture the imperfection (in plane L/500-parabolic shape)

- Gusset plates were modeled **combining elastic Beam-Column** and **FBE** (two integrations points per element)

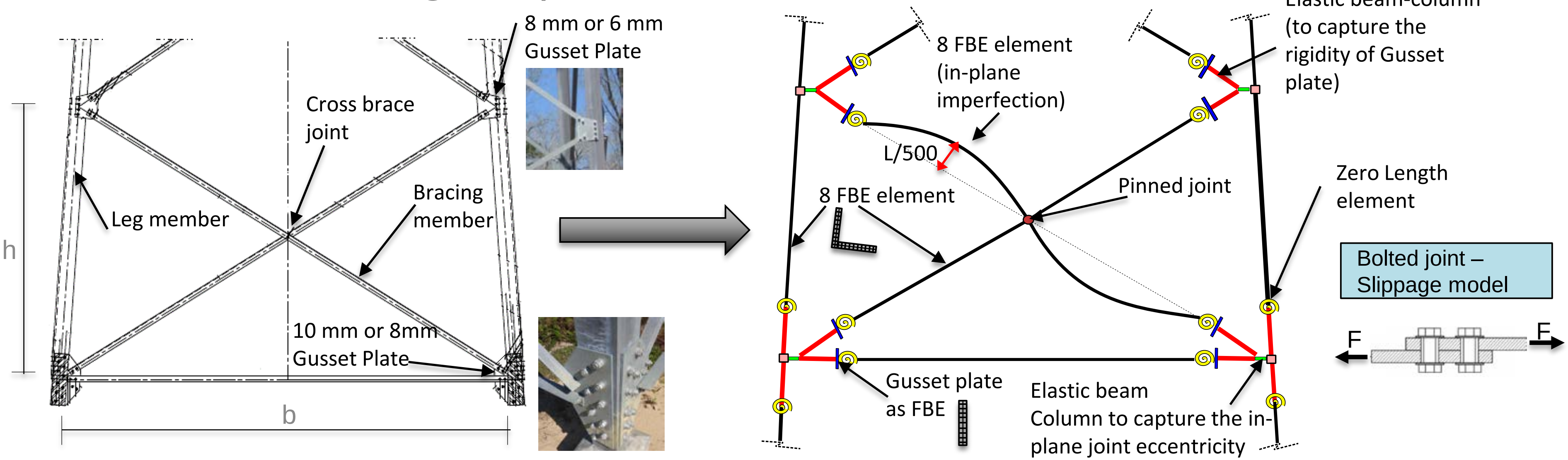
- The **material model steel02** uniaxial Giuffre Menegotto-Pinto material is used for steel fibers

- **Corotational geometric transformation** was adopted to take into account geometric nonlinearities
- Only **in-plane joint eccentricities** were considered in the modelling

The study ignored possible **local buckling effects** in the leg member cross sections

Capacity assessment of a lattice transmission tower

Numerical modeling in OpenSees of a Lattice Tower



Tower Elevation

Numerical model

- Cross brace joint, modelled with **Equal dof constraints** (translation and torsional dof)

- Gusset Plates (GP) connection modelled as:
  - Assumed as “rigid” elastic beam-column element with 10*A,10*I of the connected member;
  - The **GP modeled** with a **FBE** of **length** 2*thickness_plate and **width** based on Whitmore width (to capture out-plane resistance of the GP);
- Zero length elements were used to simulate the **slippage joint behavior** for the **axial d.o.f.** and for the **rotational d.o.f. a semi-rigid joint with linear behavior (empirical formulation) is adopted**

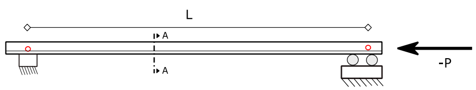
Capacity assessment of a lattice transmission tower

Hysteretic Behavior of a single angle member

Experimental test features

Member condition:

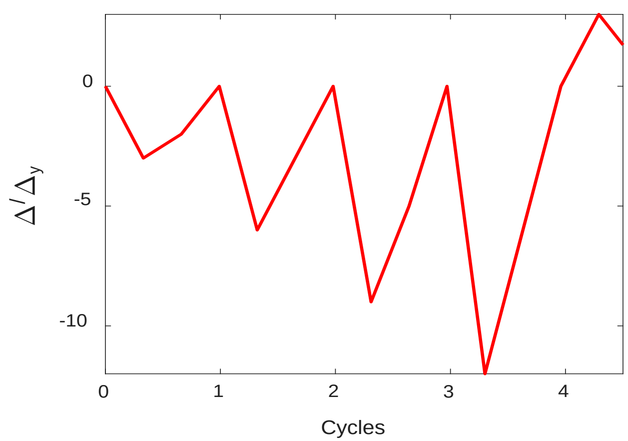
- Fixed member with angle Section L25*6
- L=1.193 m, KL/r=120, b/t=4
- Steel f_y=345MPa;



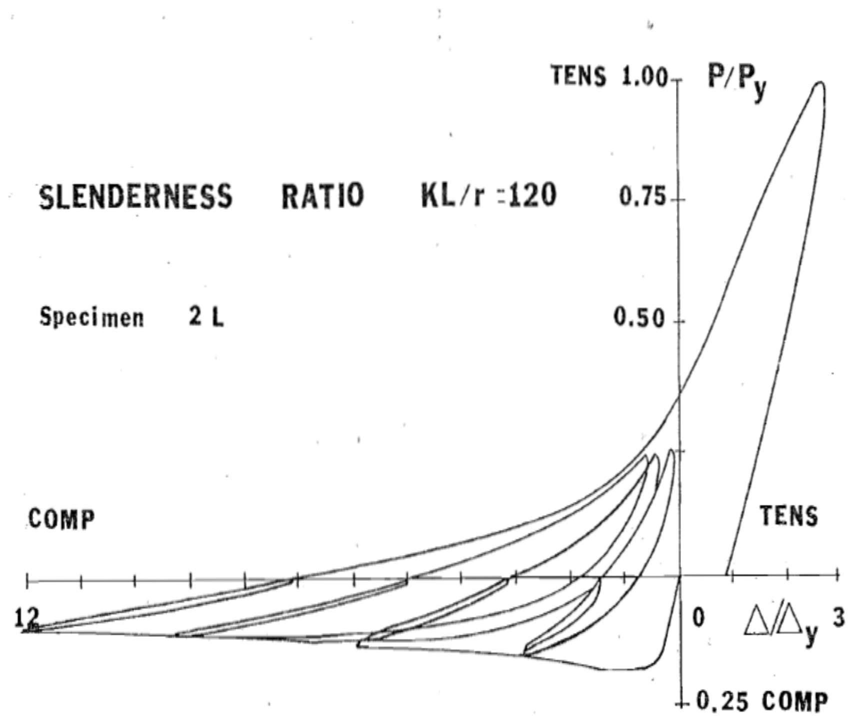
Loading Protocol:

- Formed by two deflection sequence: 1 and 2

<u>Sequence 2:</u>



Experimental Result from literature



Hysteretic Behavior of Bracing members and seismic response of braced frames with different proportions. A.K. Jain, S.C.Goel ,R.D.Hanson , Report no. UMEE 78R3 , 1978.

Capacity assessment of a lattice transmission tower

Hysteretic Behavior of a single angle member

OpenSEES Model

Numerical Simulation

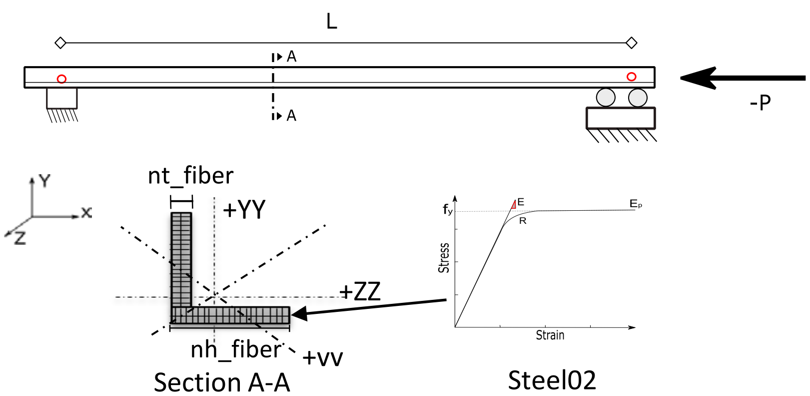
Model Features:

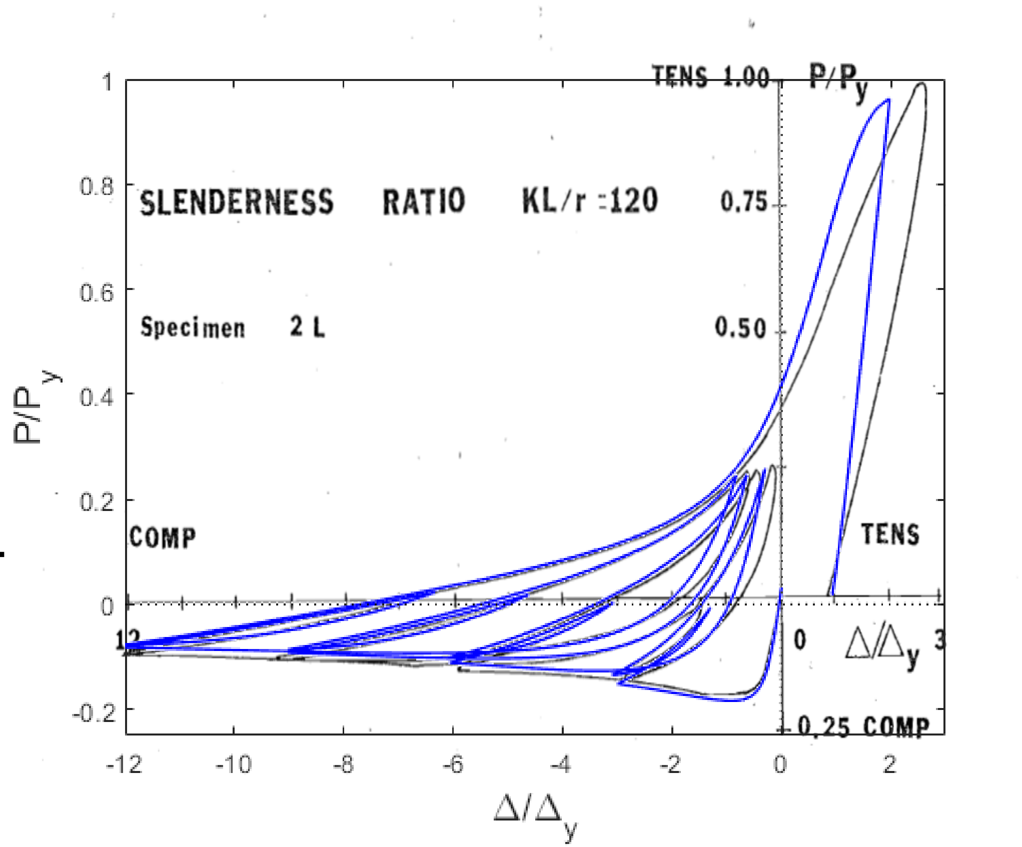
- In-plane imperfections (w/ parabolic shape e0=L/500 and **8 FBE per member**)

  -n_{t_fiber}=4; n_{h_fiber}=10

Steel Menegotto-Pinto properties:

- fy=345MPa; E=200 GPa
- b=0.1% ,R0=20, cR1=0.925, cR2=0.15
- a1=0.39 , a2=1.0, a3=0.029, a4=1.0





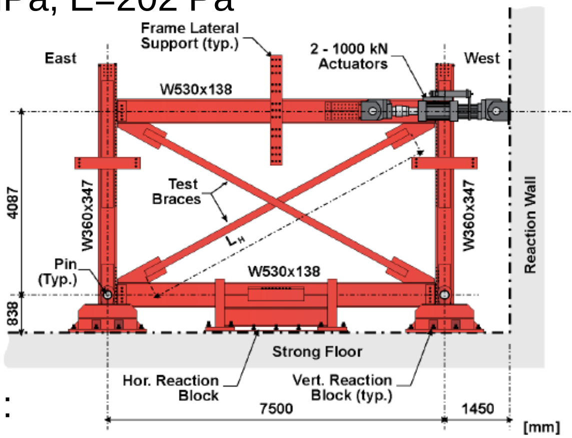
Capacity assessment of a lattice transmission tower

Hysteretic Behavior of a cross-brace panel

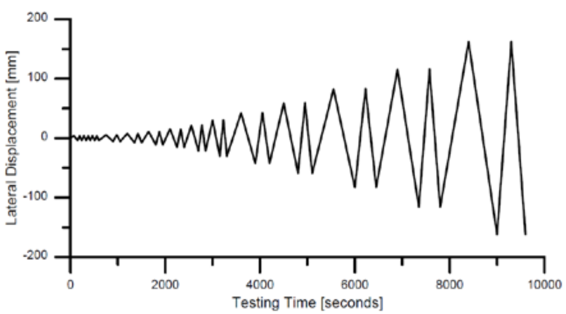
Experimental test features

Member condition:

- Test brace : L127*76x9.5; L/r=198, b/t=13.3
- Steel f_y=352 MPa; E=202 Pa



Loading Protocol:



Experimental Result from literature

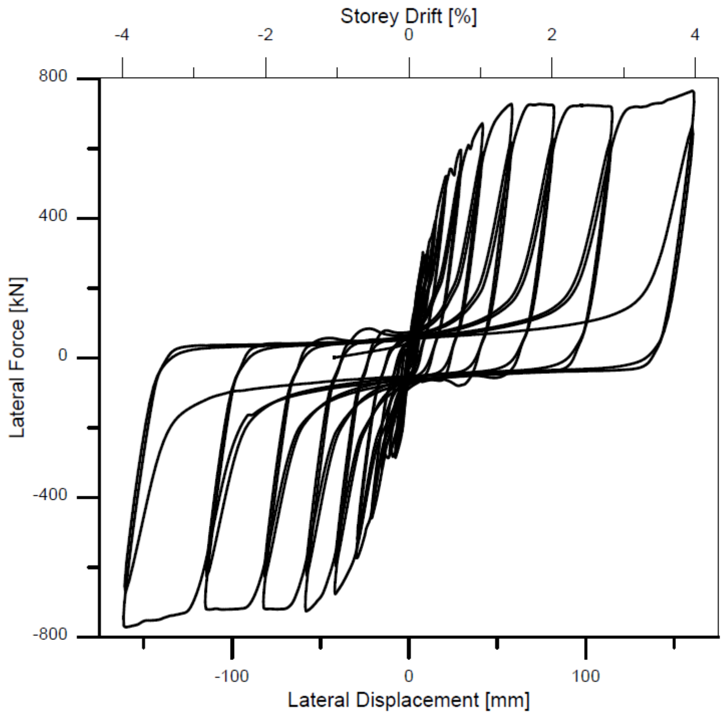


Figure 3-66: Lateral force vs. displacement hysteretic performance of plain cross brace sample

Seismic Mitigation Technique for Existing Single Storey Steel CBF Structures. T.Morrison, PhD Thesis, McGill University, 2012.

Capacity assessment of a lattice transmission tower

Hysteretic Behavior of a cross-brace panel

OpenSees Model

Model Features:

- In-plane imperfections (w/ parabolic shape $e0=L/500$ and **8 FBE per member**)

    -$n_{t\_fiber}=4$; $n_{h\_fiber}=20$;

Steel Menegotto-Pinto material model properties:

- fy=352 MPa; E=202 GPa
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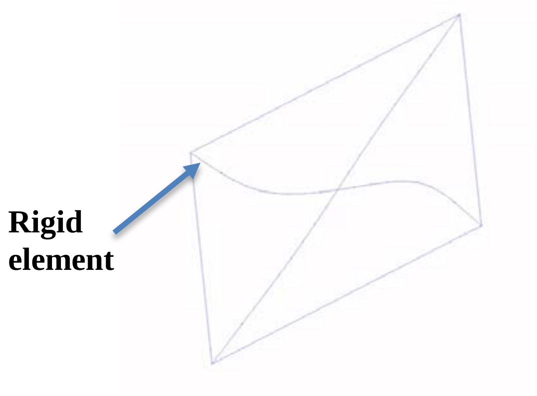
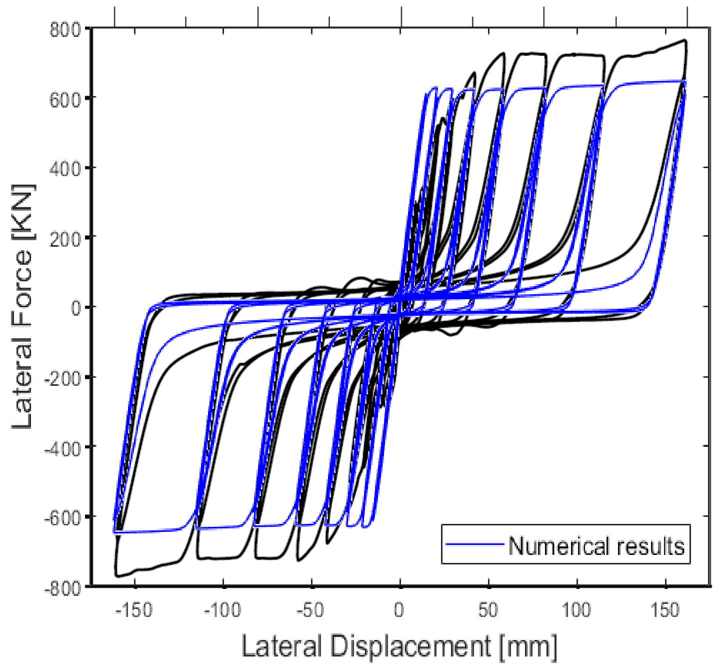
- a1=0.39 , a2=1.0, a3=0.029,a=1.0



Numerical Simulation



Lateral force vs. displacement hysteretic performance of plain cross brace sample

Capacity assessment of a lattice transmission tower

Loading Patterns Distributions considered for the Pushover Analysis

Loading patterns:

- **Uniform**

$F_i = W_i$

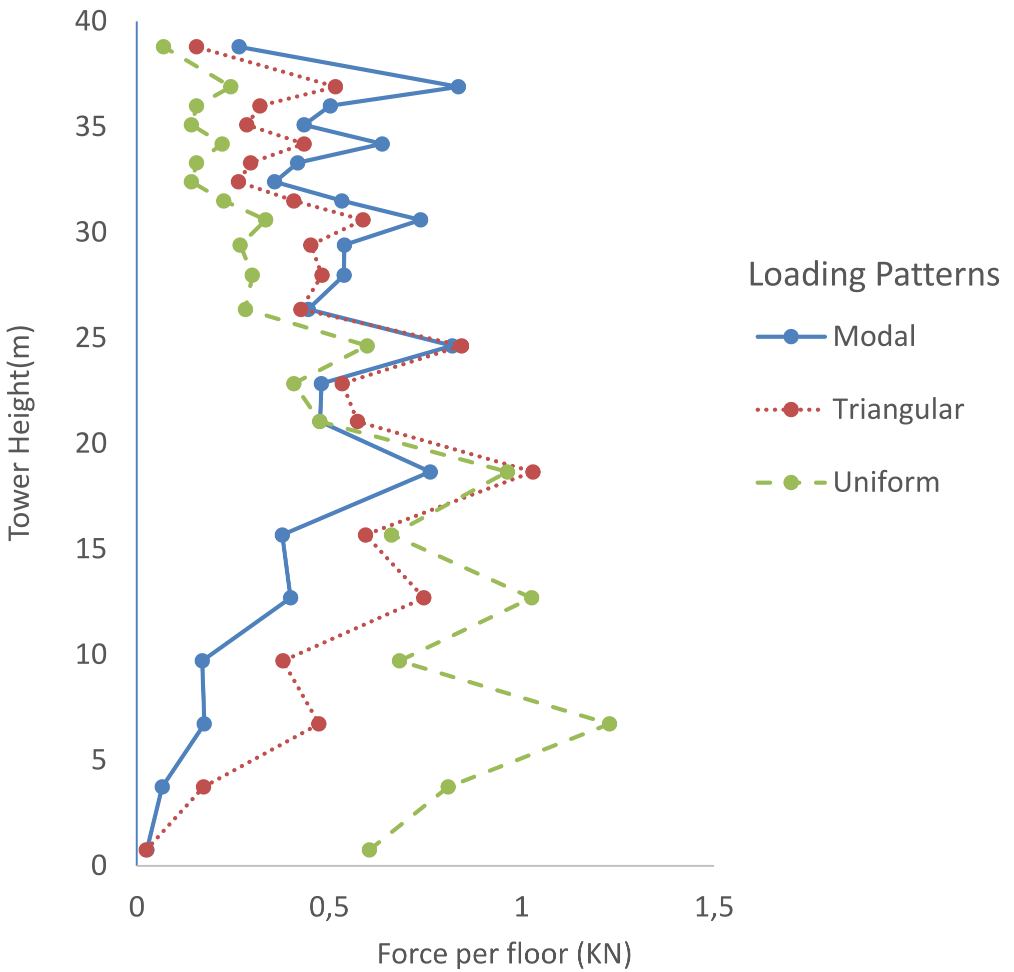
- **Triangular**

$$F_i = \frac{W_i h_i}{\sum_{l=1}^{n} W_l h_l} V_b$$

- **Modal**

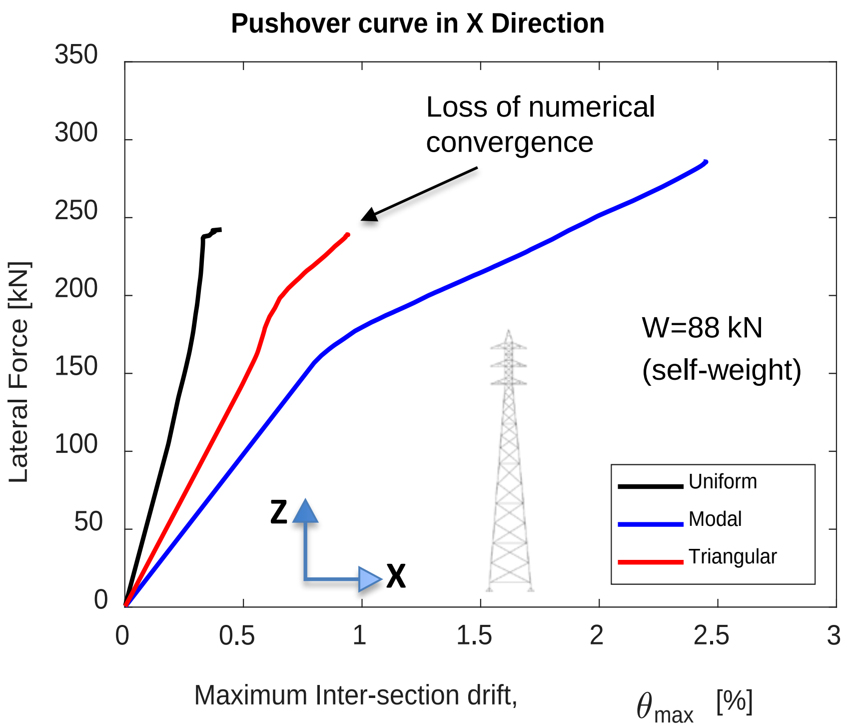
$F_i = W_i \phi_{ij}$

Total force applied to the tower scaled to: $\sum F_i$ =10 kN

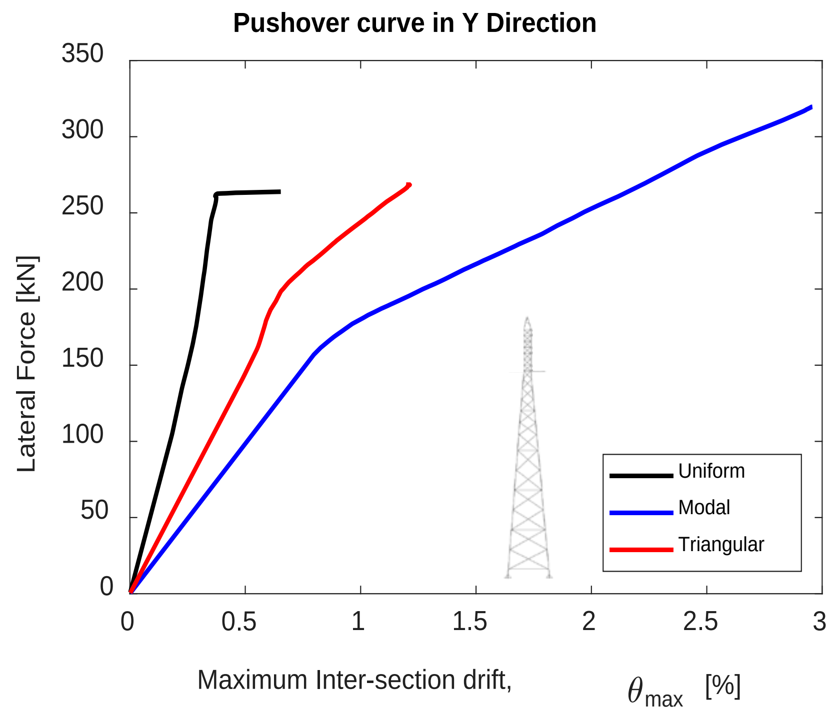


Capacity assessment of a lattice transmission tower

Pushover Curves– Model with imperfections



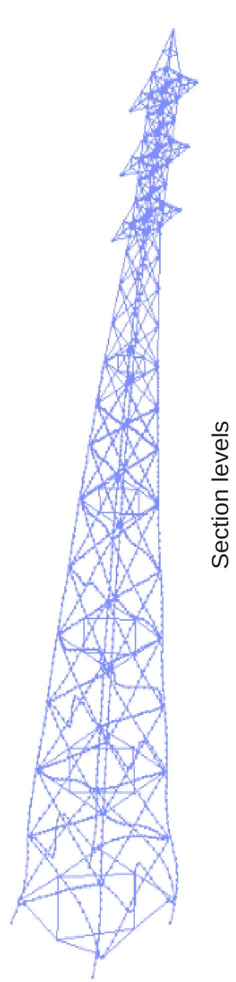
	Load Patterns		
	Modal	Triangular	Uniform
V_b (kN)	285.8	239	242.3
θ_{max} (%)	2.45	0.94	0.41
$Driftx_{max}$ (%)	1.57	0.67	0.27



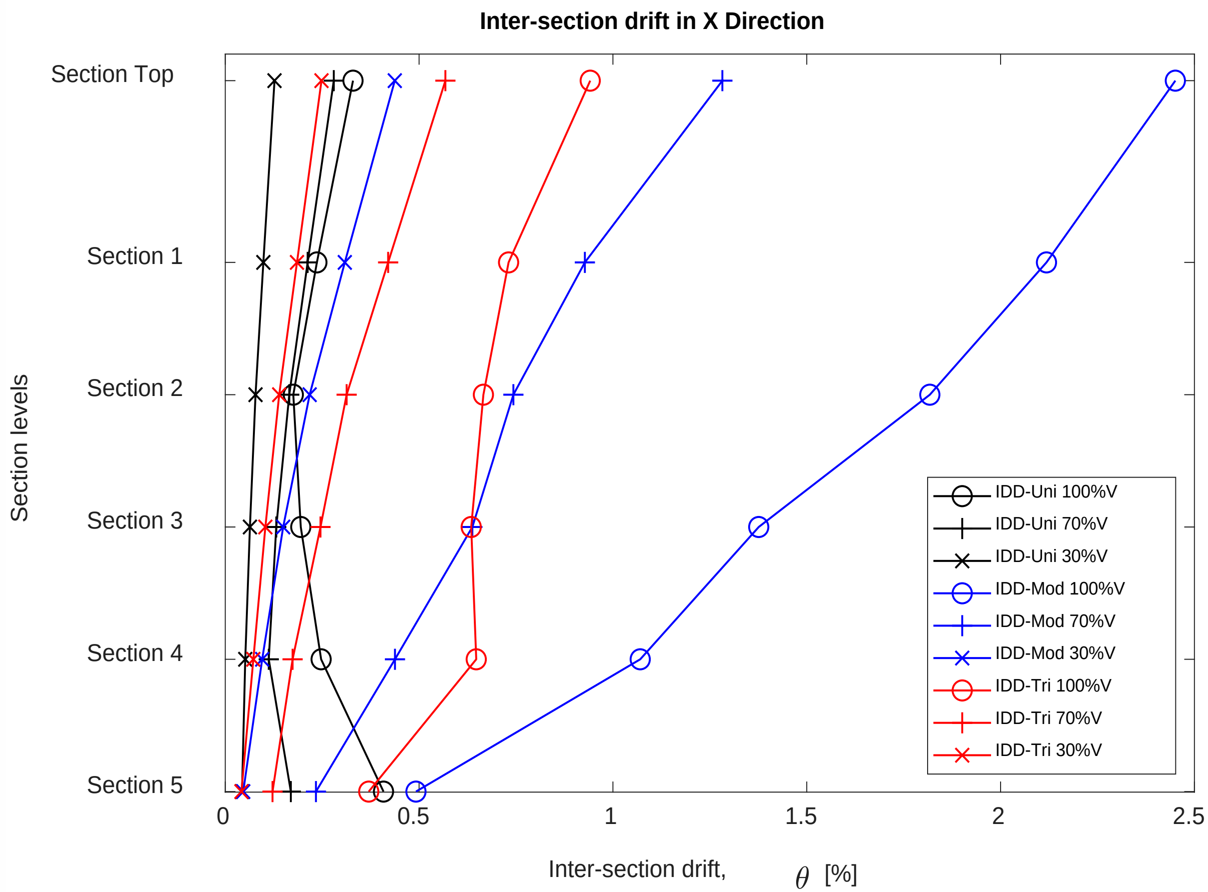
	Load Patterns		
	Modal	Triangular	Uniform
V_b (kN)	319.6	268.4	263
θ_{max} (%)	2.95	1.21	0.65
$Drifty_{max}$ (%)	1.86	0.89	0.31

Capacity assessment of a lattice transmission tower

Pushover Curves– Model with imperfections



Uniform - collapse mode



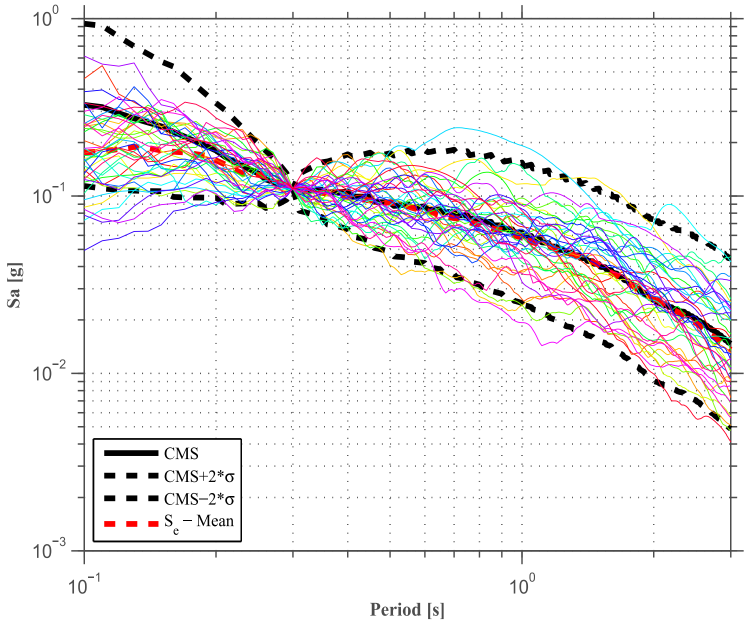
Seismic Collapse Risk assessment of a lattice Transmission Tower

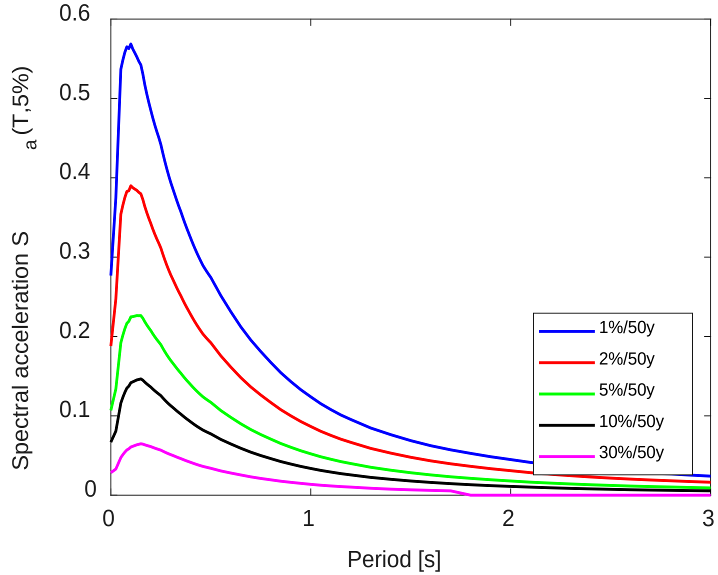
Selection and Scaling of Ground Motion Records

- Assess the **annual rate of collapse** of the isolated tower

$$\lambda_{collapse} = \int \phi(\frac{\ln(IM_i/\mu)}{\beta}) \, |d\lambda_{IM}(x)|$$

- Selection and Scaling of GM records based on Conditional Mean Spectrum (SelEQ Framework, Macedo 2017)





- Conditional Mean Spectrum for Sa(T1=0.30 s ,5%) ;
- Probability of exceedance **10% in 50y** →T=475 y;
- Soil Type C assumed
- A total of **40 GM with 2 components (NGA West2)**

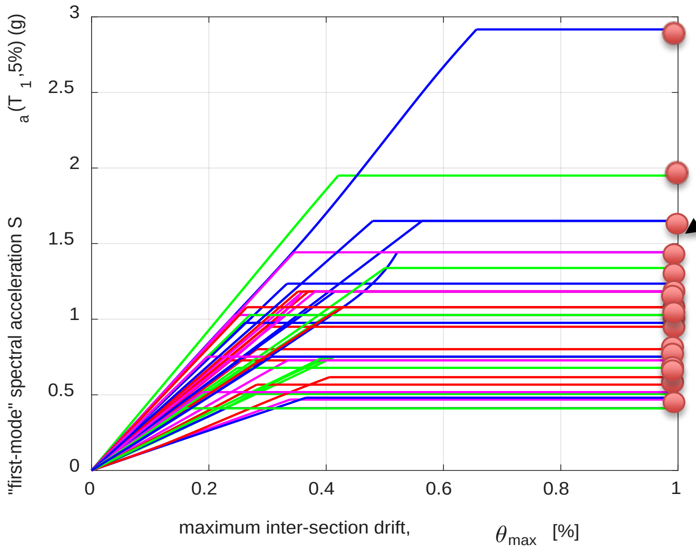
- Median Uniform Hazard Spectrum for different probability of exceedance obtained from PSHA

Seismic Collapse Risk assessment of a lattice Transmission Tower

Collapse data obtained through an Incremental Dynamic Analysis

Selected IM:
Sa(T_1=0.30 s ,5%) in the x direction

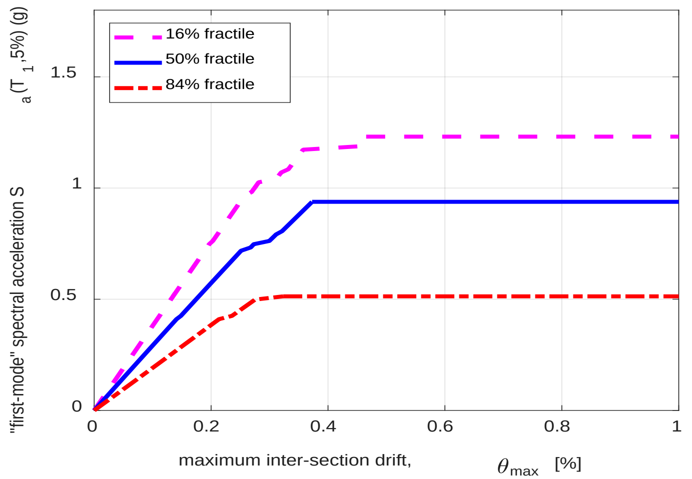
Selected EDP:
Maximum Inter-Section drift: $\theta_{max} = \sqrt{\theta_x^2 + \theta_y^2}$



Preliminary results

Other considerations for the **Dynamic Analysis:**

- Mean properties (JCSS Probabilistic Model code)
- Rayleigh Damping model ($C = \alpha M + \beta K_{ini}$)
    - ξ=5% viscous damping
    - α and β (f_1=3,27 Hz and f_2=13.01Hz)

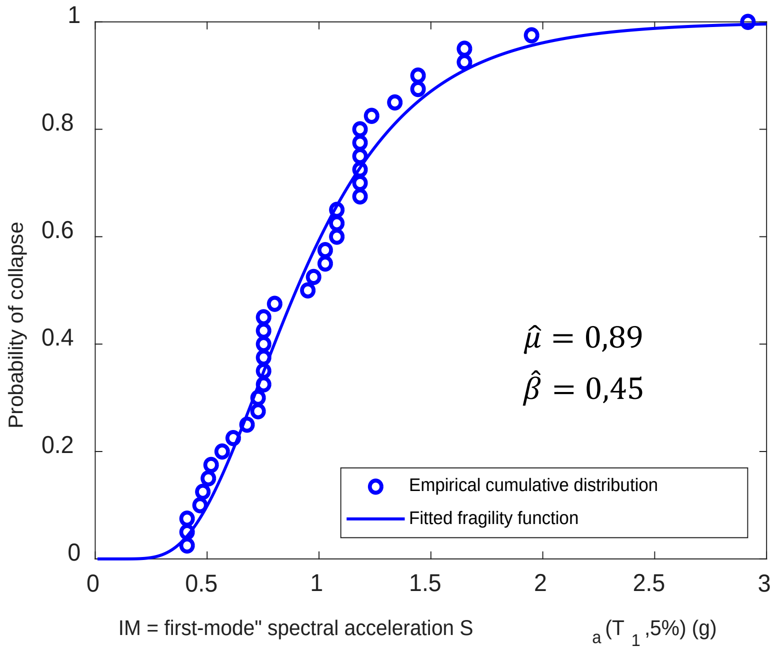


Seismic Collapse Risk assessment of a lattice Transmission Tower

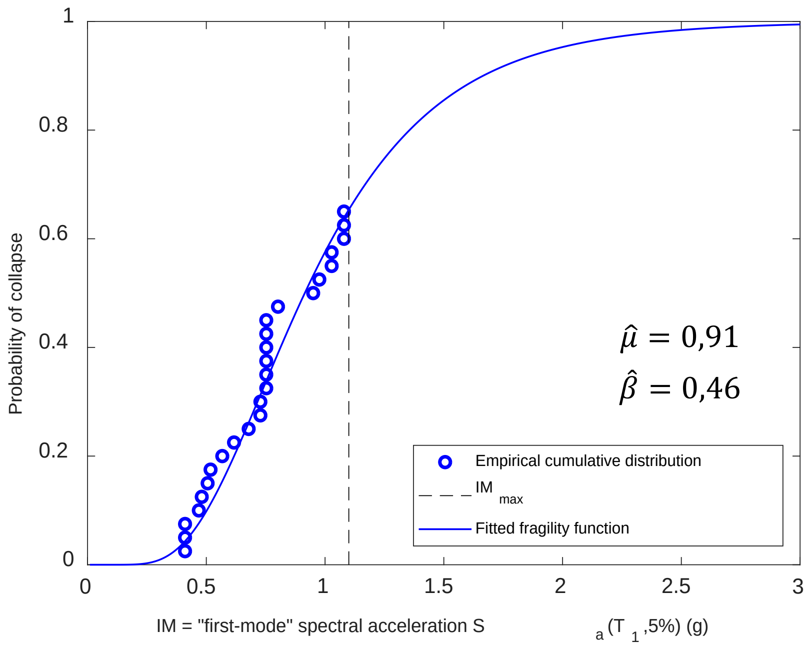
Collapse data obtained through an Incremental Dynamic Analysis

- Fragility function fitting with the **method of moments**
- Assumed a lognormal distribution

$$Pcollapse = \phi(\frac{\ln(IM_i/\mu)}{\beta})$$



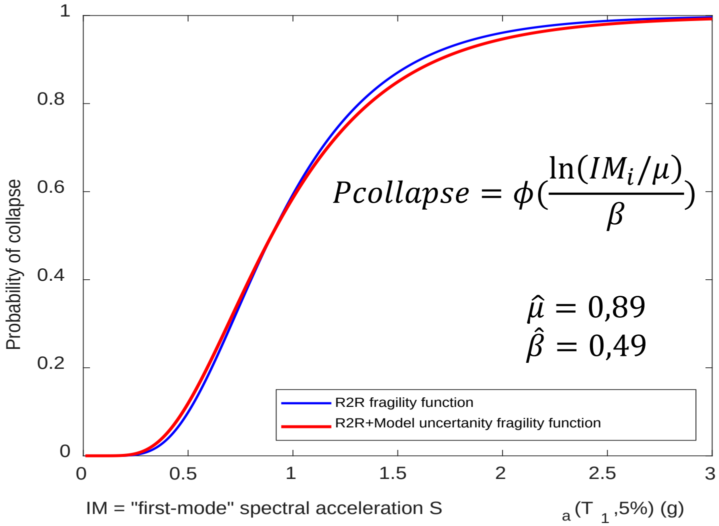
Truncated IDA at **IM_{max}=1.1g**

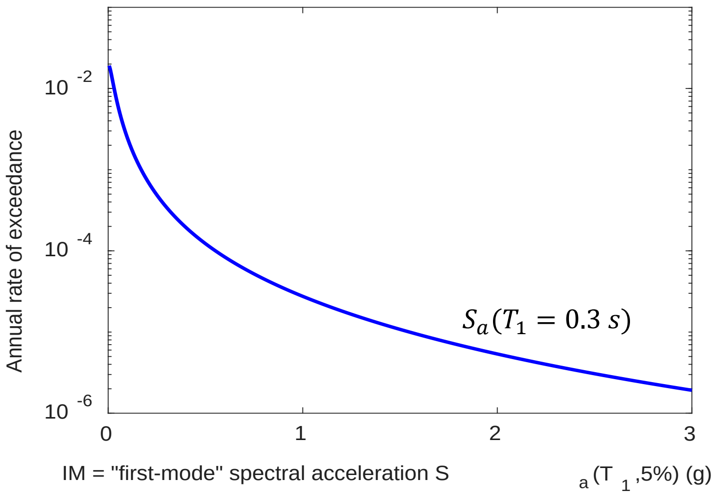


Seismic Collapse Risk assessment of a lattice Transmission Tower

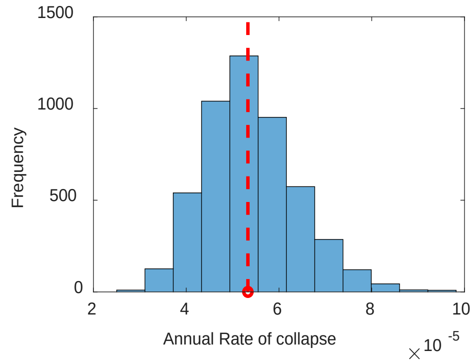
Assessment of the annual collapse rate as a risk metric

- Fragility function considering additional source of uncertainty
- **Model uncertainty** assumed at $\beta = 0.2$ based on paper Full scale tests of Transmission Towers Riera, J.D .et al. (1990) – Cigre document





$$\lambda_{collapse} = \int \phi(\frac{\ln(IM_i/\mu)}{\beta})\ |d\lambda_{IM}(x)|$$



	Uncertainty type	
	R2R	R2R+Model
$\hat{\lambda}_{collapse}$	$5.42 * 10^{-5}$	$5.89 * 10^{-5}$
C.O.V (Parametric Bootstrap)	19%	-
Pcollapse in 50y (%)	$[0.37 - 0.16]$ $\pm 2\sigma$	0.29 (mean value)

Conclusion and Future Developments

Main Conclusions

*The single angle and cross-bracing panel **numerical models*** have been capable of simulating the strength and stiffness degradation from experimental results.

*Slippage effects produce an **increase in the tower lateral deformation** that is dependent on the load pattern (factor 1.0-2.0) but they do not seem to affect the **ultimate load capacity**. The **triangular load pattern** have shown the better prediction in the obtained maximum inter-section drift when compared with the dynamic analysis results.*

The structure in general displays a fragile behavior (overstrength factor very limited). The effect of model uncertainty seems of minor importance in the assessment of the annual rate of collapse for the Earthquake hazard.

Future Developments

*Development of **fragility curves** with different procedures for Earthquake and Wind Hazard (more focus given to the Wind Hazard) for the tower+cable system.*

*Assessment of the **wind risk** of the isolated tower and a simplified “tower+cable” system considering model uncertainties.*

Thank you!