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REPUTATION RESOURCES RESULTS

Analysis and Design of Vibration-Suppressing Systems for Stay Cables

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Analysis and Design of Vibration-Suppressing Systems for Stay Cables

- Introduction
  - Known cases of vibrations
  - Phenomena and reasons for cable vibrations
- Methods for assessment of cable vibrations
  - Practical formulae for assessment of vibration likelihood
  - Analytical methods for response predictions
- Design Considerations
- Case Study: Ironton-Russell Bridge
  - Cable vibration analysis
  - Vibration-suppressing system

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Bridges with Cable Vibration Problems

Second Severn Crossing, UK
Completed 1996

Veterans Memorial Bridge,
W. Virginia/Ohio, USA, 1990

Fred Hartman Bridge, USA
Completed 1995

Faro Bridge, Denmark
Completed 1984
(photograph Olaf Niederlein)

Phenomena and reasons for cable vibrations

Sources of Cable Vibrations

- Aerodynamic and aeroelastic instabilities
  - vortex shedding (low amplitudes – 0.5~1*D*)
  - rain-wind vibrations (large amplitudes - 1~2 m)
  - dry inclined galloping (large amplitudes - 1~2 m)

- Other probable sources of vibrations
  - direct wind buffeting on cables (order of 1*D* once in 100 years)
  - bridge vortex shedding & wind buffeting (large amplitudes - 1~2 m)
  - vehicles & pedestrian (typically small)

Note: *D* – cable diameter

Phenomena and reasons for cable vibrations

Why stay cables vibrate?

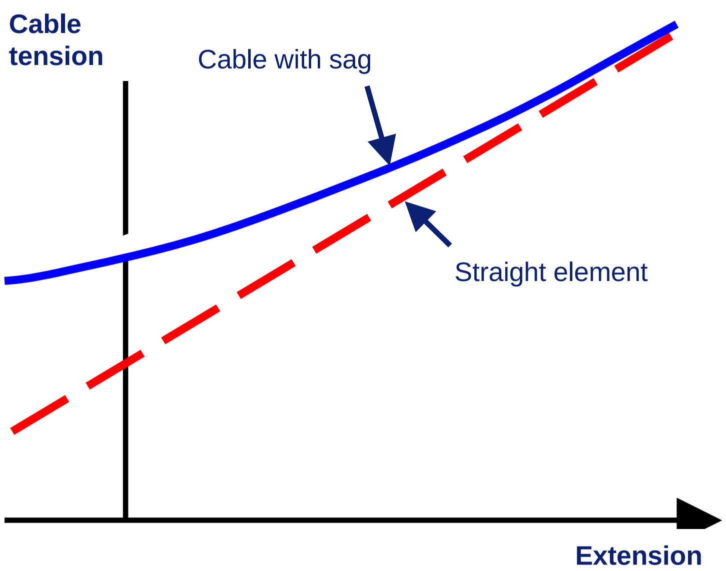
- Low structural damping

  - structural damping - extremely low 0.03~0.1% of critical, where aerodynamic damping may be significant, on long cables at high wind speeds it could be more than 1%

- Low mass – typically 50 - 150 kg/m

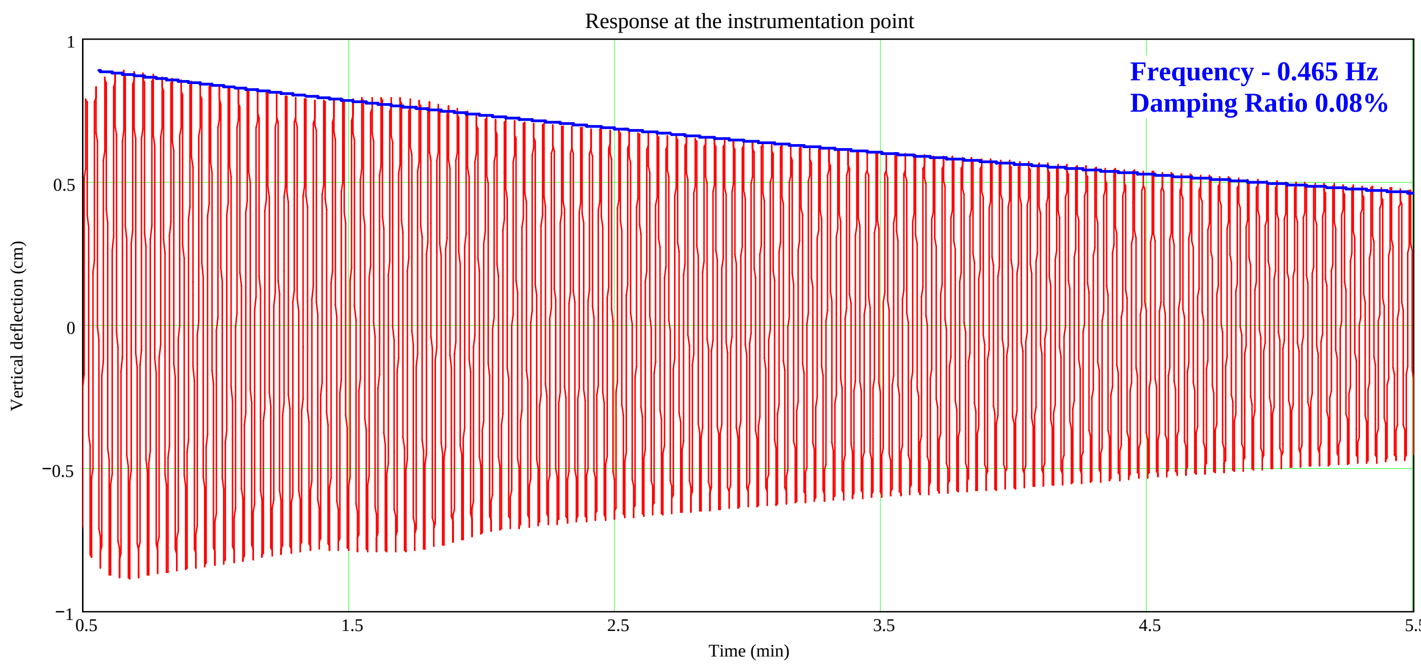
- Non-linear structural behavior

Structural properties



Non-linearity of cable stiffness

On long cables, low frequencies, and very low structural damping



Governing stability parameter

- Scruton number S_c

  (also called mass-damping parameter)

  μ – mass per liner length
  ζ – structural damping
  ρ – air density
  D – outside diameter

$$S_c = \frac{\mu\zeta}{\rho D^2}$$

- Without damper devices S_c is very low

$$S_c \sim 1$$

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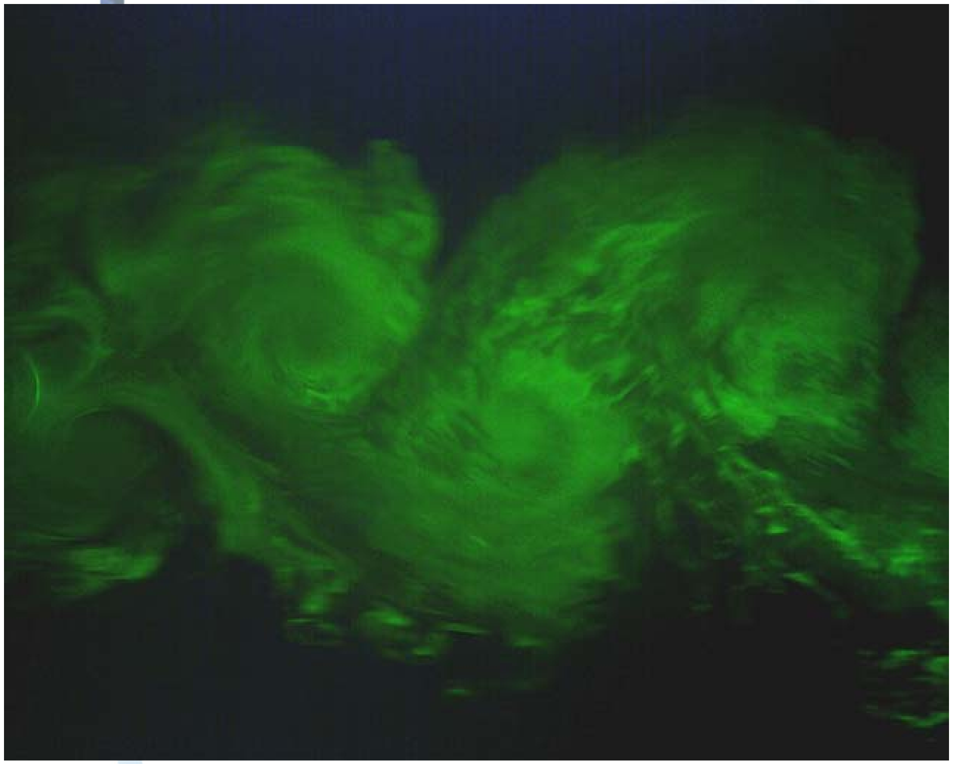
What do we know of the excitation phenomena

- Vortex shedding
- Rain-wind induced vibrations
- Direct wind buffeting
- Dry inclined cable galloping
- Motion-induced vibrations

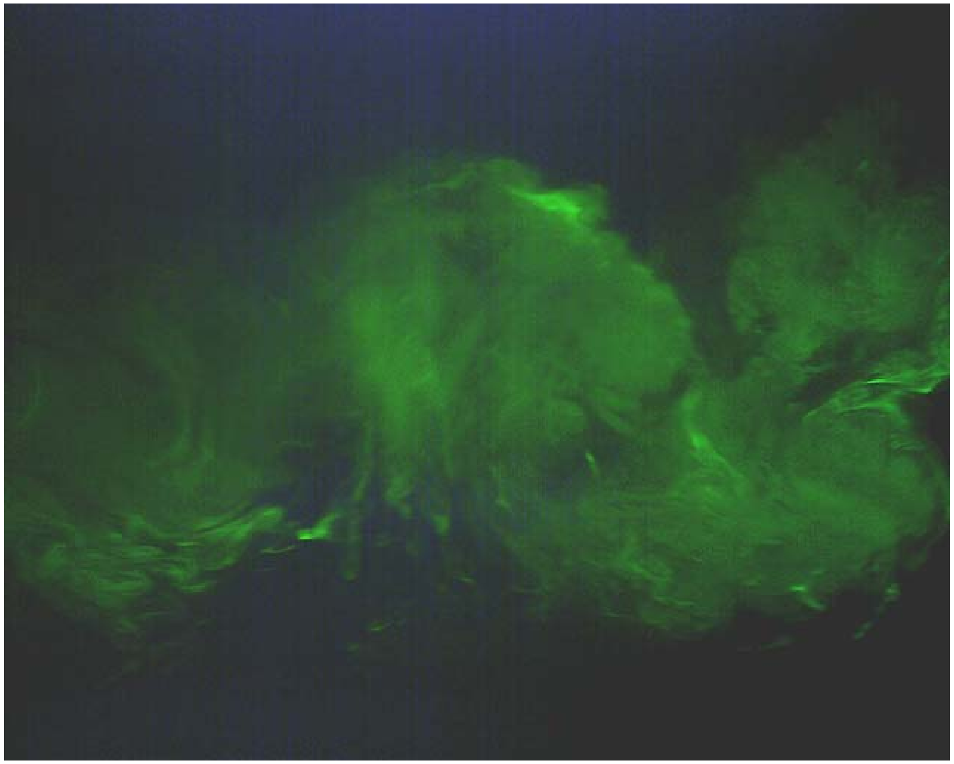
Generally understood (Vortex shedding, Rain-wind induced vibrations, Direct wind buffeting)

Complex phenomena, focus of current research (Dry inclined cable galloping, Motion-induced vibrations)

Instabilities - Vortex Shedding



2s mode



2p mode

[data from Sherbrooke University – courtesy of Dallaire and Laneville, 2005]

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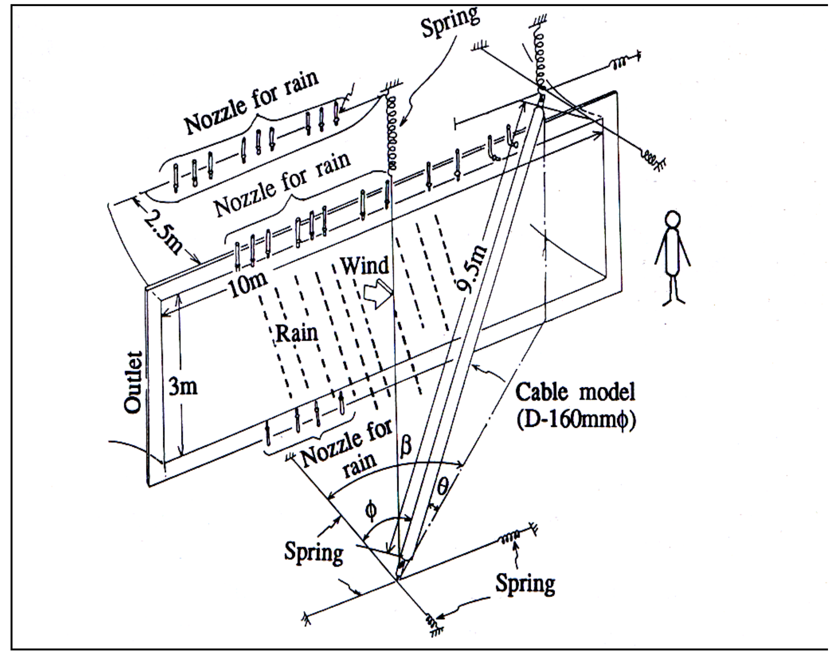
Wind tunnel test including water rivulet effects

Cables vibration test with moving water rivulet



Wind tunnel testing on cable model under rain and wind simulations

[courtesy Professor M. Matsumoto]

Methods for assessment of cable vibrations

- Practical formulae for assessment of vibration likelihood
- Analytical methods for response predictions

Practical formulae for assessment of vibration likelihood

- Onset speed of any instability $V_{onset} > V_{criteria}$
- Vortex shedding responses **would be small** if $S_c > 2.5$
- Rain-wind oscillations **will not occur** if $S_c > 10$
- Other instabilities $S_c > ?$

Analytical Methods for Estimation of Cable Vibrations

- Vortex Shedding - Ruscheweyh 1986, ESDU 96030
- Rain-Wind Vibrations – not available
- Dry Inclined Galloping – Macdonald 2005
- Tower-Cable-Deck Excitations

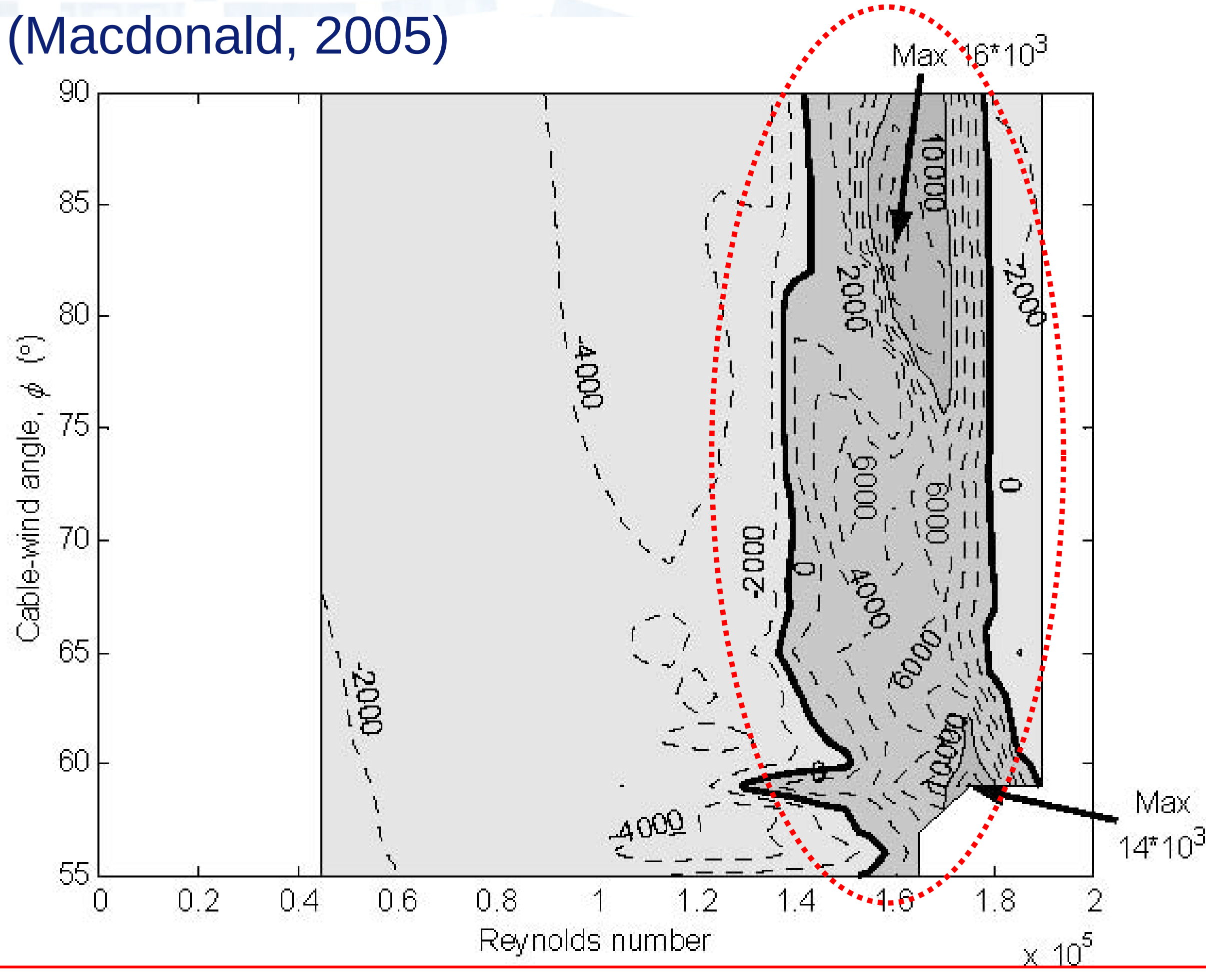
  Fujino et al 1993, Lilien and Pinto da Costa 1994

  Virlogeux 1998, SETRA-LCPC 2002,

  Direct Simulations in Time Domain

Dry Inclined Galloping

- Experimental results, turbulence corrected (Macdonald, 2005)



Critical Reynolds number region of instability: $1.4x10^5 < Re < 1.8x10^5$

Dry Inclined Galloping

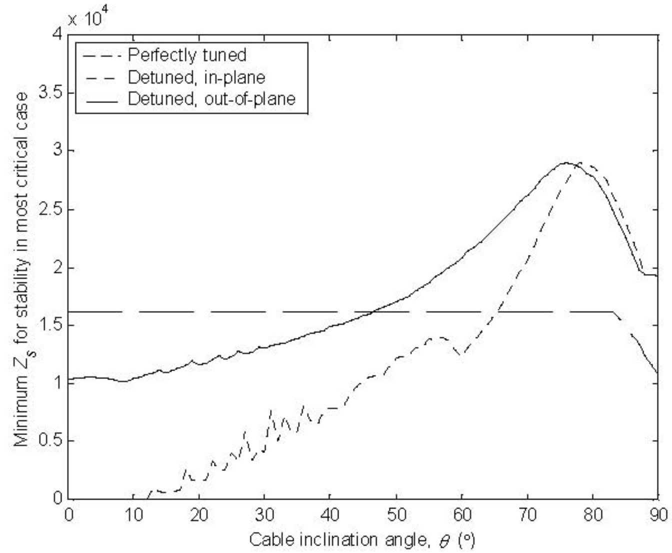
- Critical onset velocity $$V_{gal} = \frac{\mathrm{Re} \cdot \nu}{D}$$

ν = 1.45 x 10^{-5} m^2/sec – air viscosity

Re = 1.4 x 10^5 - lowest instability boundary

- Minimum damping $$\xi_{str} = \frac{Z_s \rho \nu}{\mu f_n}$$

(Macdonald 2005)



Tower-Cable-Deck Interaction

- Laboratory tests and full-scale measurements confirm excitation of stay cables due motions of deck and towers (Andersen at al 1999, Macdonald 2000)

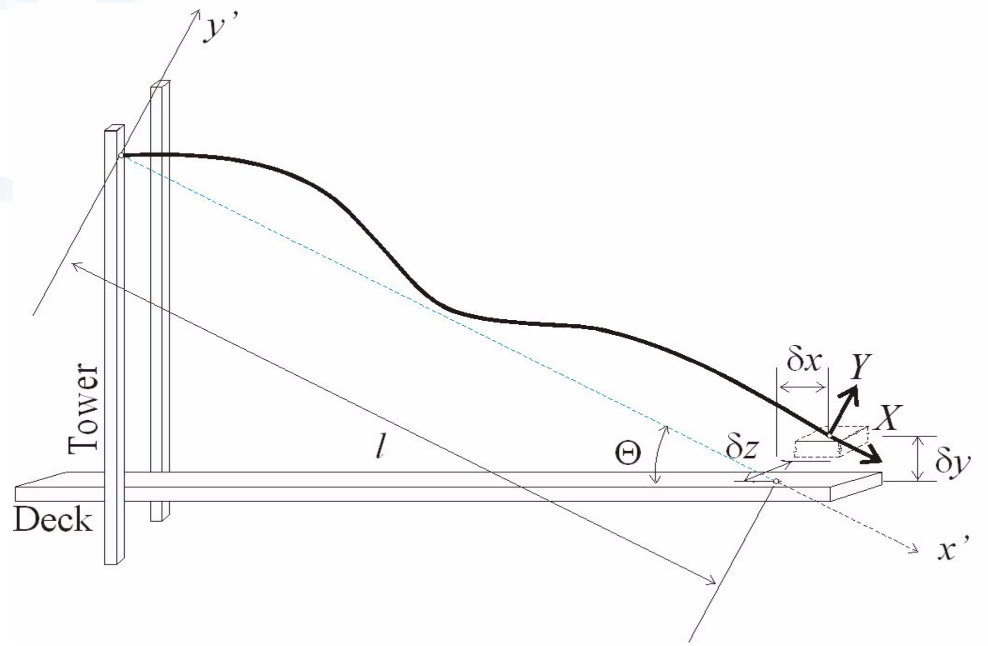
- These interactions are a consequence of frequency similarity between global bridge modes and the fundamental modes of the stay cables

Tower-Cable-Deck Interaction

- This excitation mechanism can cause:
  - Fatigue of the cable stays
  - Discomfort of users on the bridge
  - Interruption of the normal bridge operations
  - Failures
- All vibration sources must be assessed during the design of the bridge.

Tower-Cable-Deck Interaction

Dynamic Response of Cables



- **Anchorage Displacements**

Along the cable $X_m = \left| \left(x_m^t - x_m^b \right) \right| a_m$

Lateral $Y_m = \left| \left(y_m^t - y_m^b \right) \right| a_m$

- a_m are generalized displacement amplitudes obtained from numerical predictions of operational responses to wind, traffic, pedestrian, etc.

Tower-Cable-Deck Interaction

Dynamic Response of Cables

Modal frequency ratio $r_m = \frac{\Omega_m}{\omega_1}$

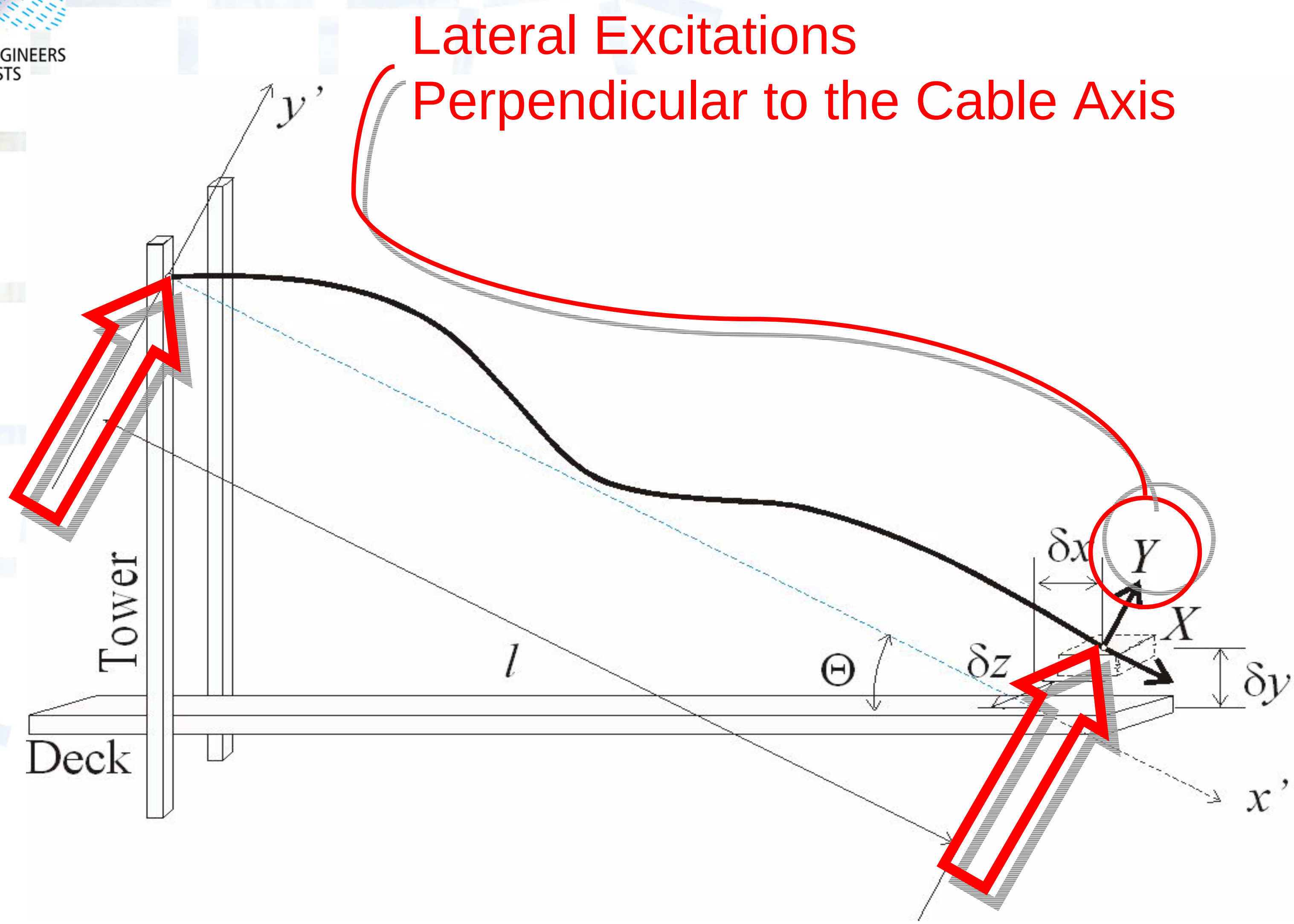
Fundamental frequency of cable ω_1

m^{th} modal frequency of bridge (estimated from FEA) Ω_m

Total damping ratio (ξ_A = aero, ξ_S = structural)

$$\xi_T = \xi_A + \xi_S = \left[\frac{\beta \rho U D C_D + c}{2 \mu \omega_1} \right]$$

Motion-Induced Cable Vibrations



Motion-Induced Cable Vibrations

Excitation Perpendicular to the Cable Axis at Primary Resonance: $r_m = 1$

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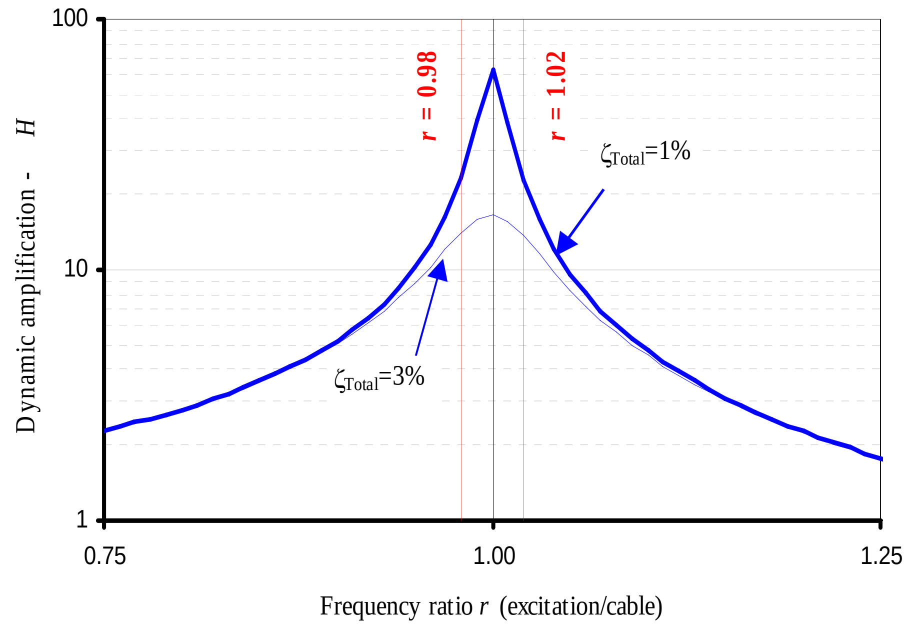
Peak Modal Response

$$A_{k,mn} = Y_{mn} \cdot \frac{2r_{mn}^2}{\pi k^3} \cdot H\left(\xi_{T,k}, \frac{r_{mn}}{k}\right)$$

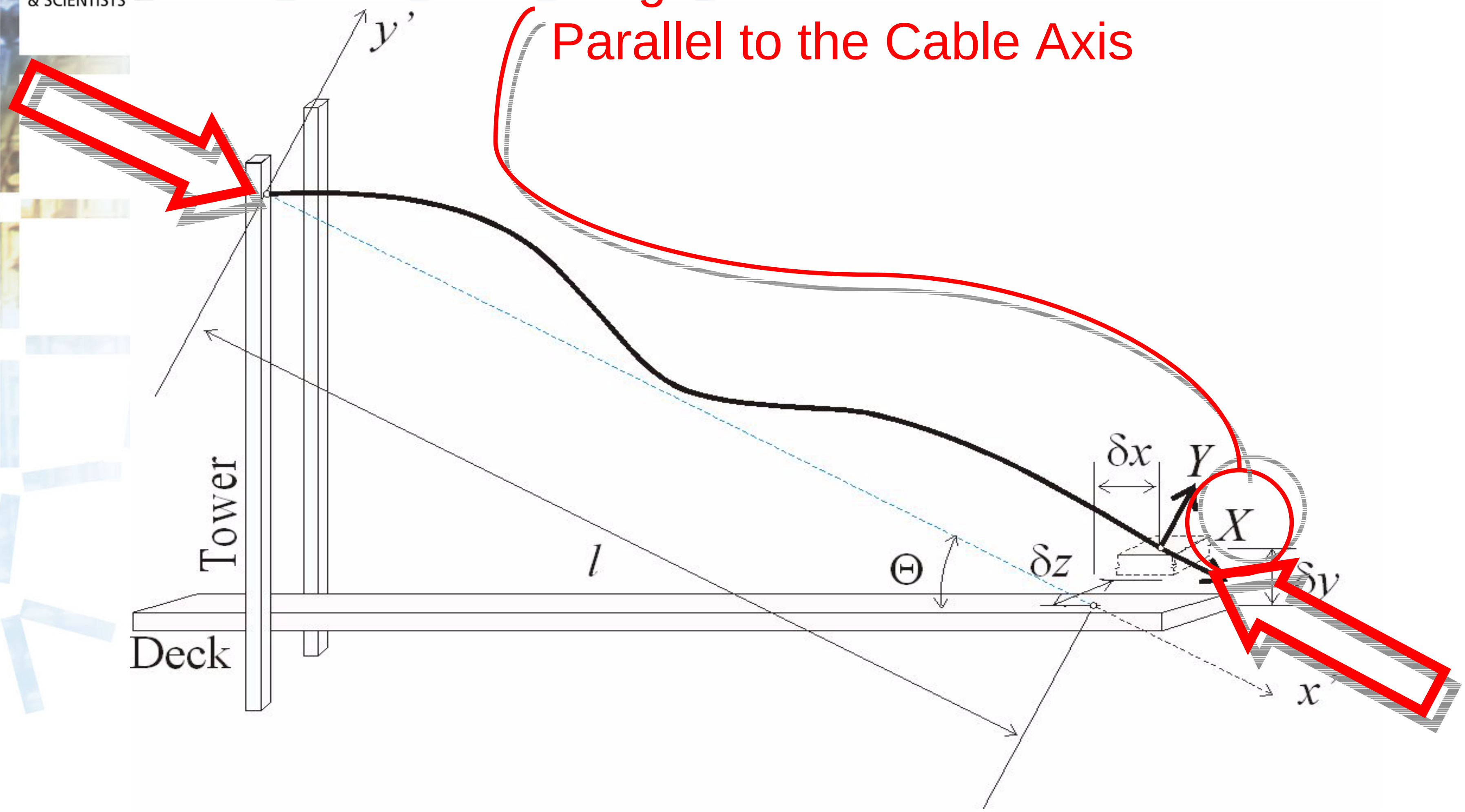
Mechanical Admittance

$$H\left(\xi_{T,k}, \frac{r_{mn}}{k}\right) = \frac{1}{\sqrt{\left(1-\frac{r_{mn}^2}{k^2}\right)^2 + \left(\frac{2\xi_{T,k}\, r_{mn}}{k}\right)^2}}$$



Parametric Excitation

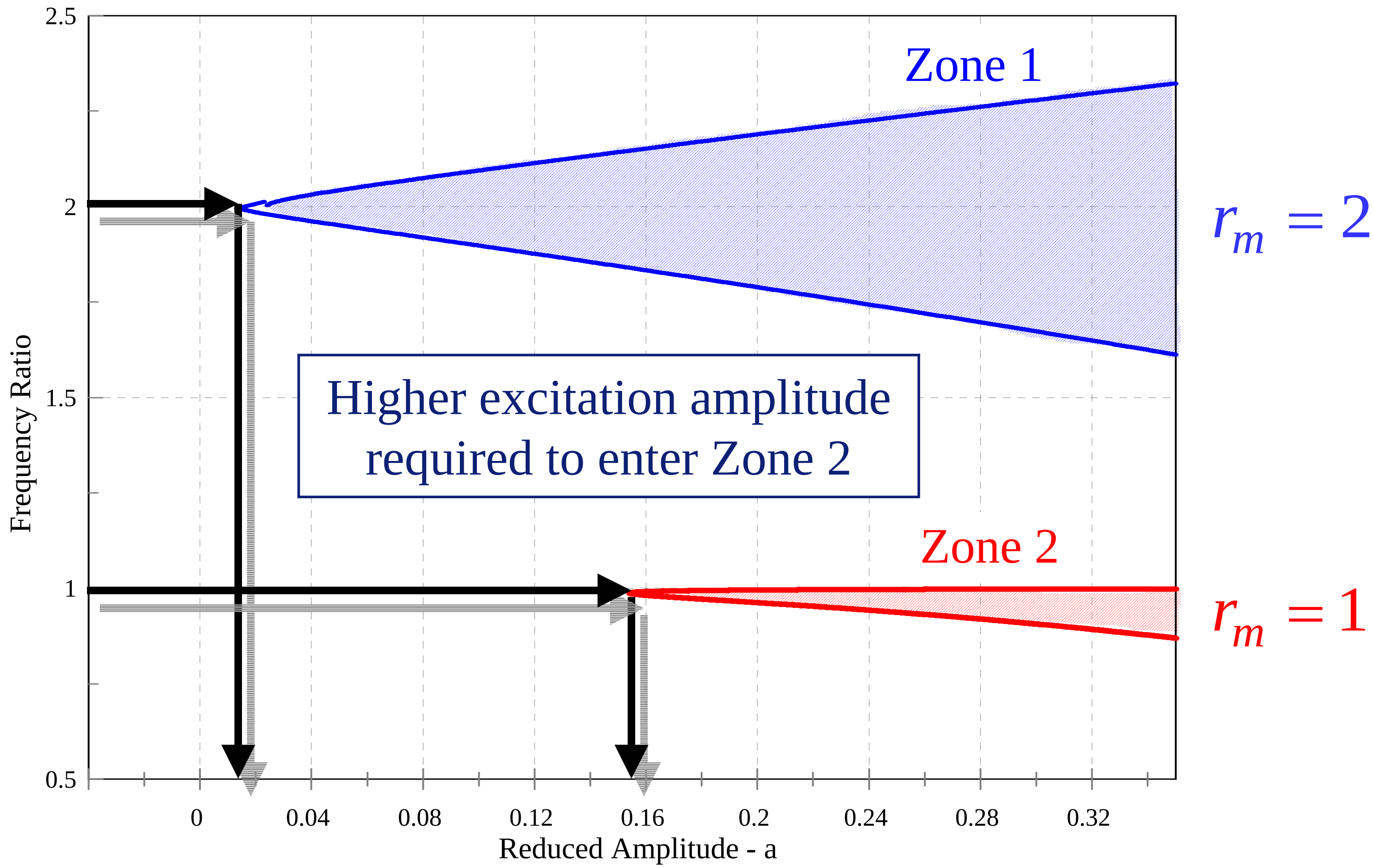
Longitudinal Excitations
Parallel to the Cable Axis



Parametric Excitation

Excitation Parallel to the Cable Axis at Parametric Resonance: $r_m = 1, 2$

Parametric instability regions (Nayfeh and Mook 1979)



Parametric Excitation

Excitation Parallel to the Cable Axis at Parametric Resonance: $r_m = 1, 2$

- Although instabilities are possible, there is clear case identified on an existing bridge.
- The excitation amplitudes (anchorage displacements) required for triggering cable vibrations could be large. High winds, over prolonged time durations would be required to attain an instability condition.
- The increased aerodynamic damping at high wind speeds is expected to prevent such instabilities from reaching significant amplitudes

Design Considerations

- Cable vibration could be controlled via
  - aerodynamic modifications – effective for rain-wind vibration
  - increased damping – has limits of how much damping could be added, damping is frequency dependent
  - frequency detuning – cross-ties effective only for close to in-cable-plane vibrations

- Selection of Vibration Suppressing System
  - depends on the geometry and dynamics of given bridge
  - cost effectiveness & contractor capabilities
  - maintenance and long-term performance

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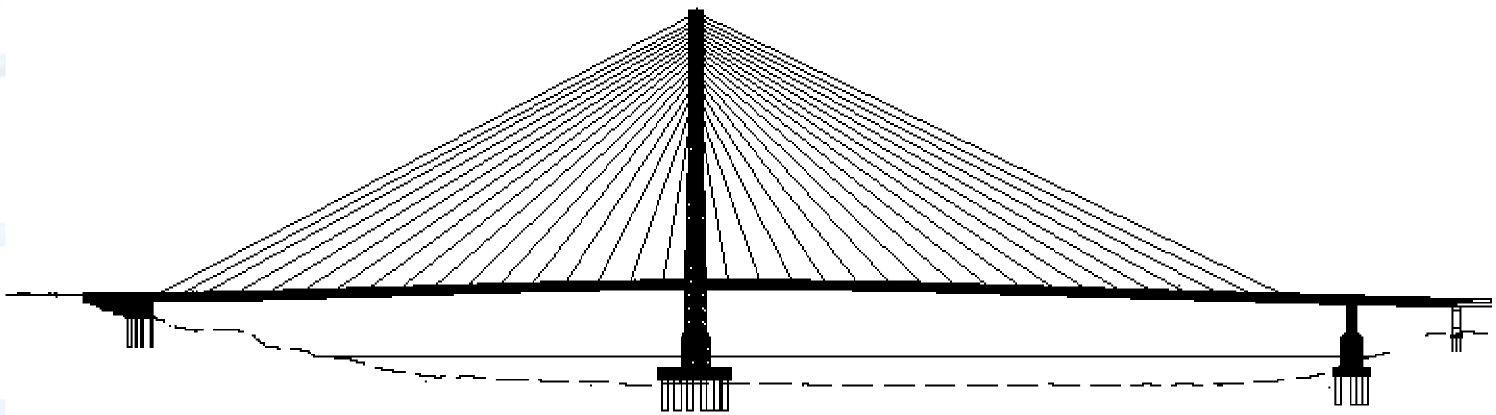
Case Study:
the Single Tower Schema
for Ironton-Russell Bridge

The Ironton-Russell Bridge



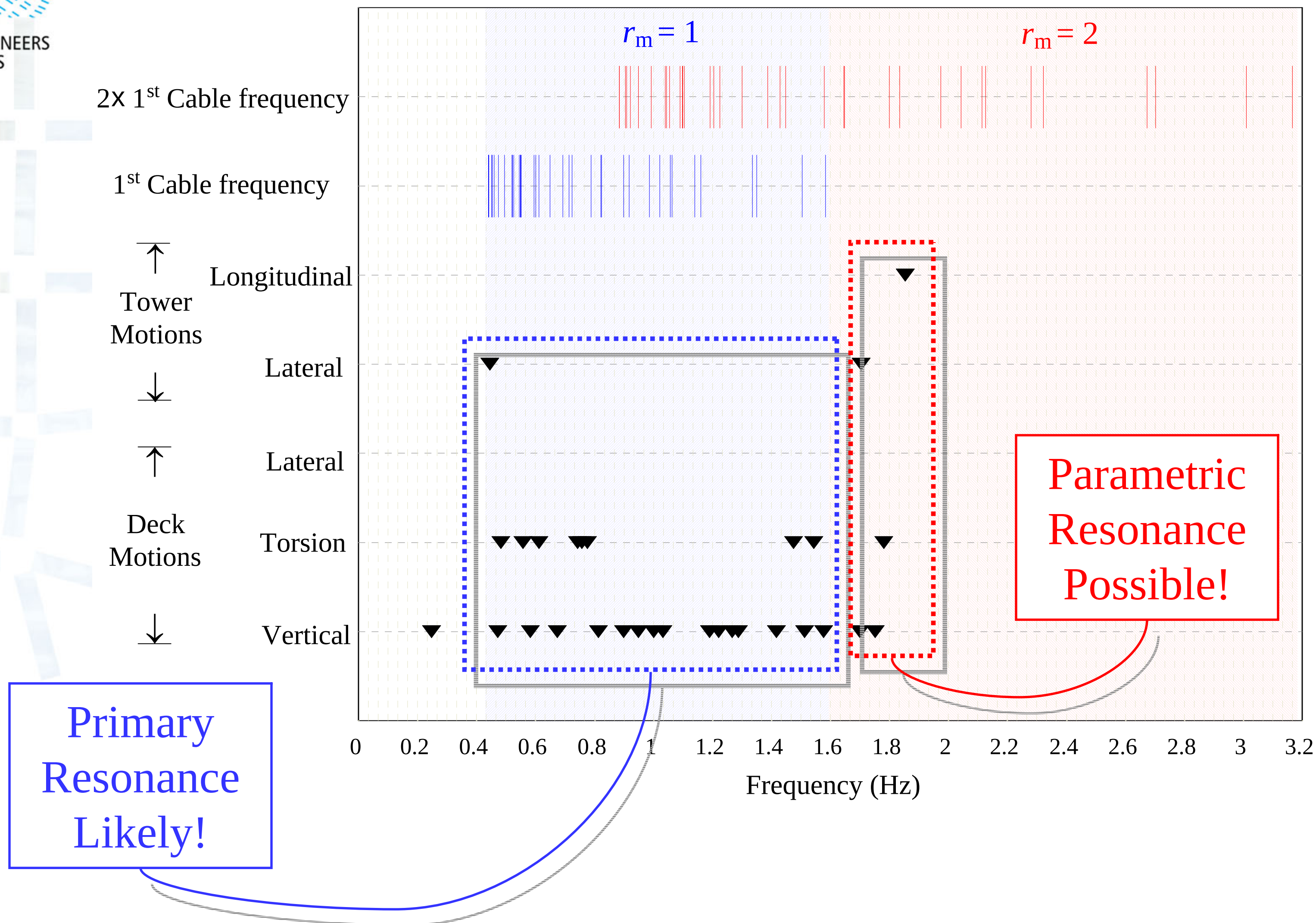
Schema of the proposed Ironton Russell Bridge

- To span the Ohio River between Ironton, Ohio and Russell, Kentucky
- 35 cables in each cable plane - 18 main span, 17 back span.
- Longest stay cables in North America: 291 m
- Design by Michael Baker Jr.
- *Given bridge configuration was not awarded for construction*

Cable Vibration Analysis

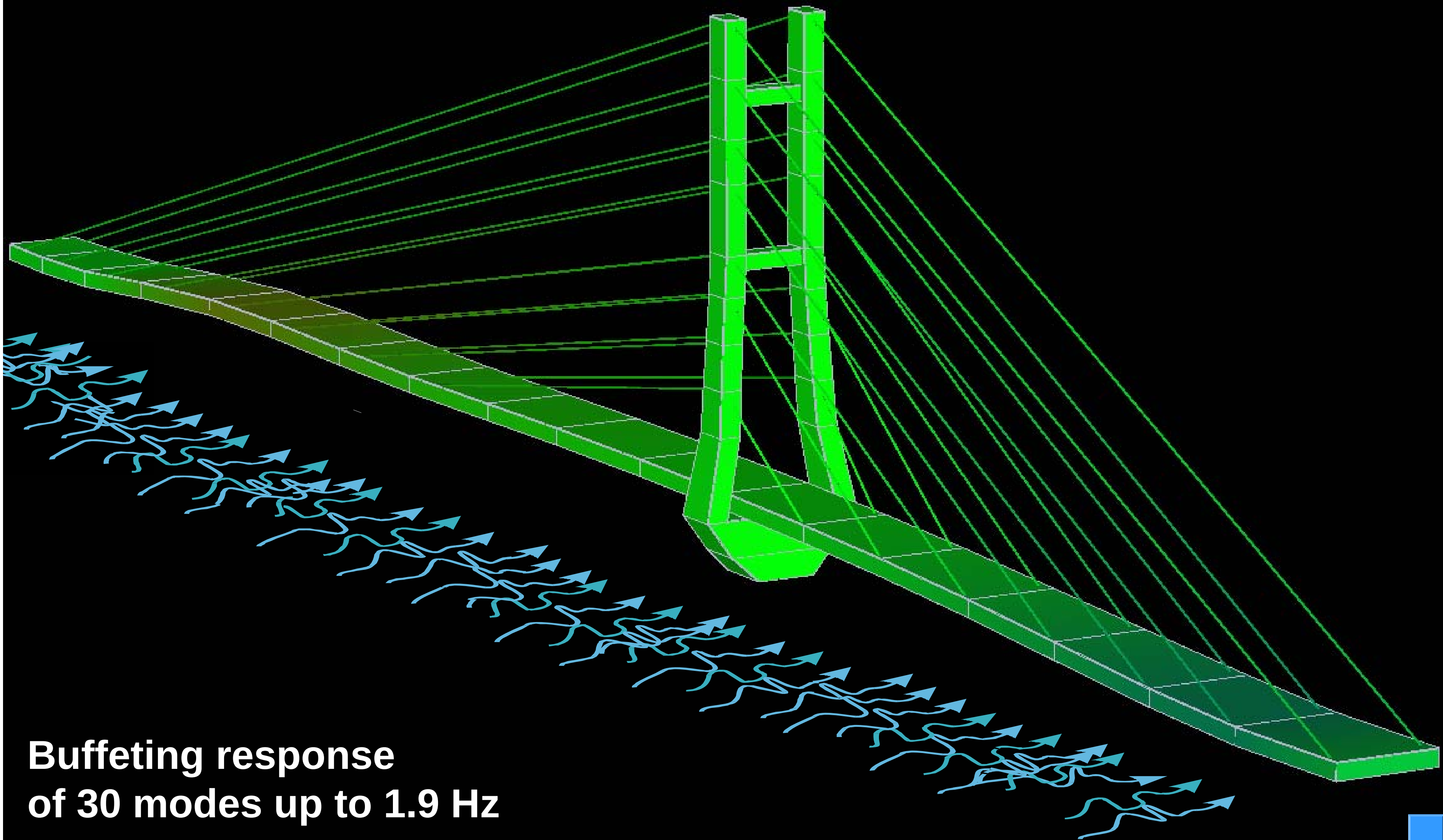
Modal Frequency Comparison

Ironton-Russell Bridge



Buffeting Response Analysis

Ironton-Russell Bridge



Buffeting response of 30 modes up to 1.9 Hz

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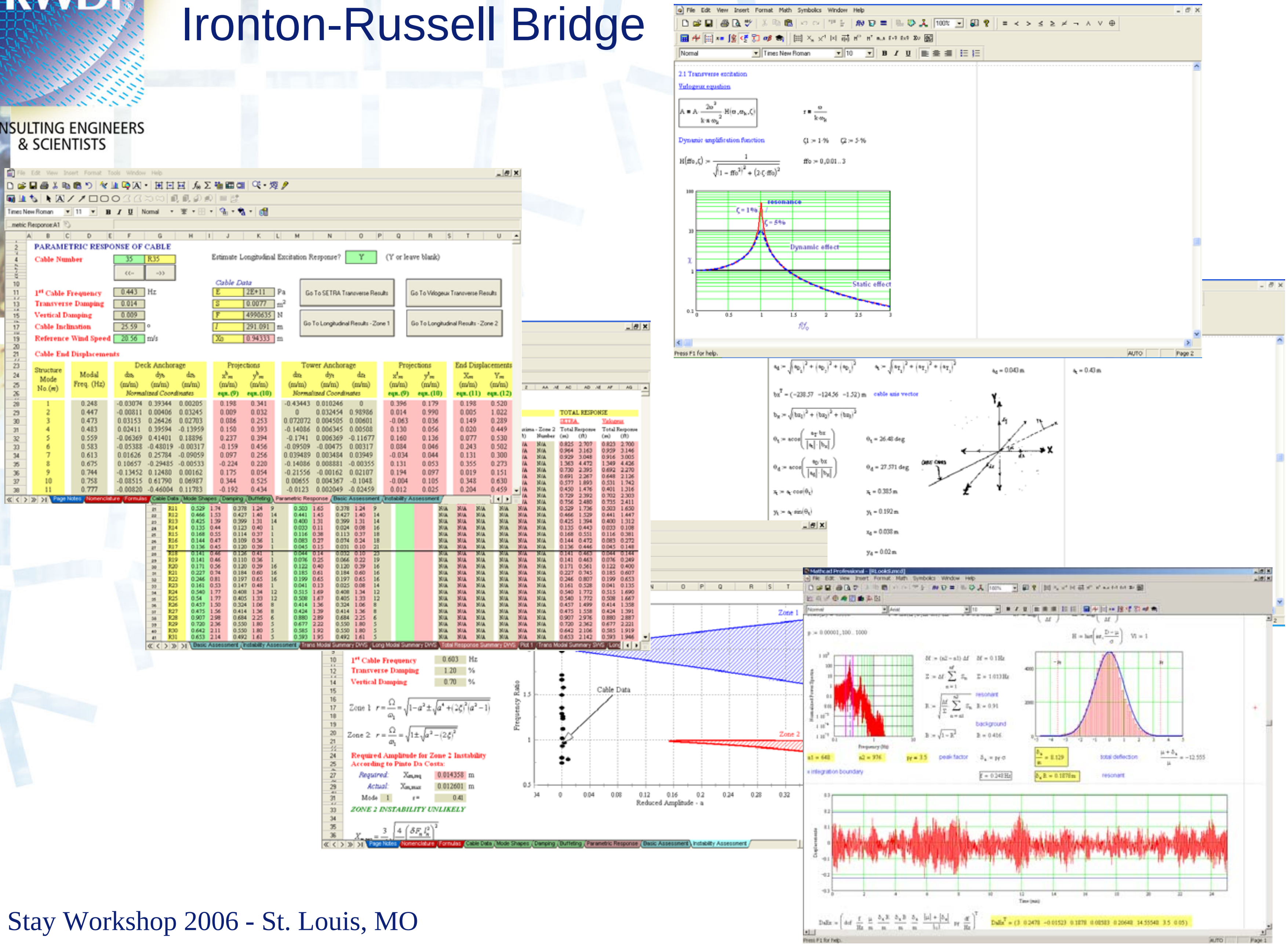
Response Analysis

Ironton-Russell Bridge



Comparison of Virlogeux and SETRA formulae

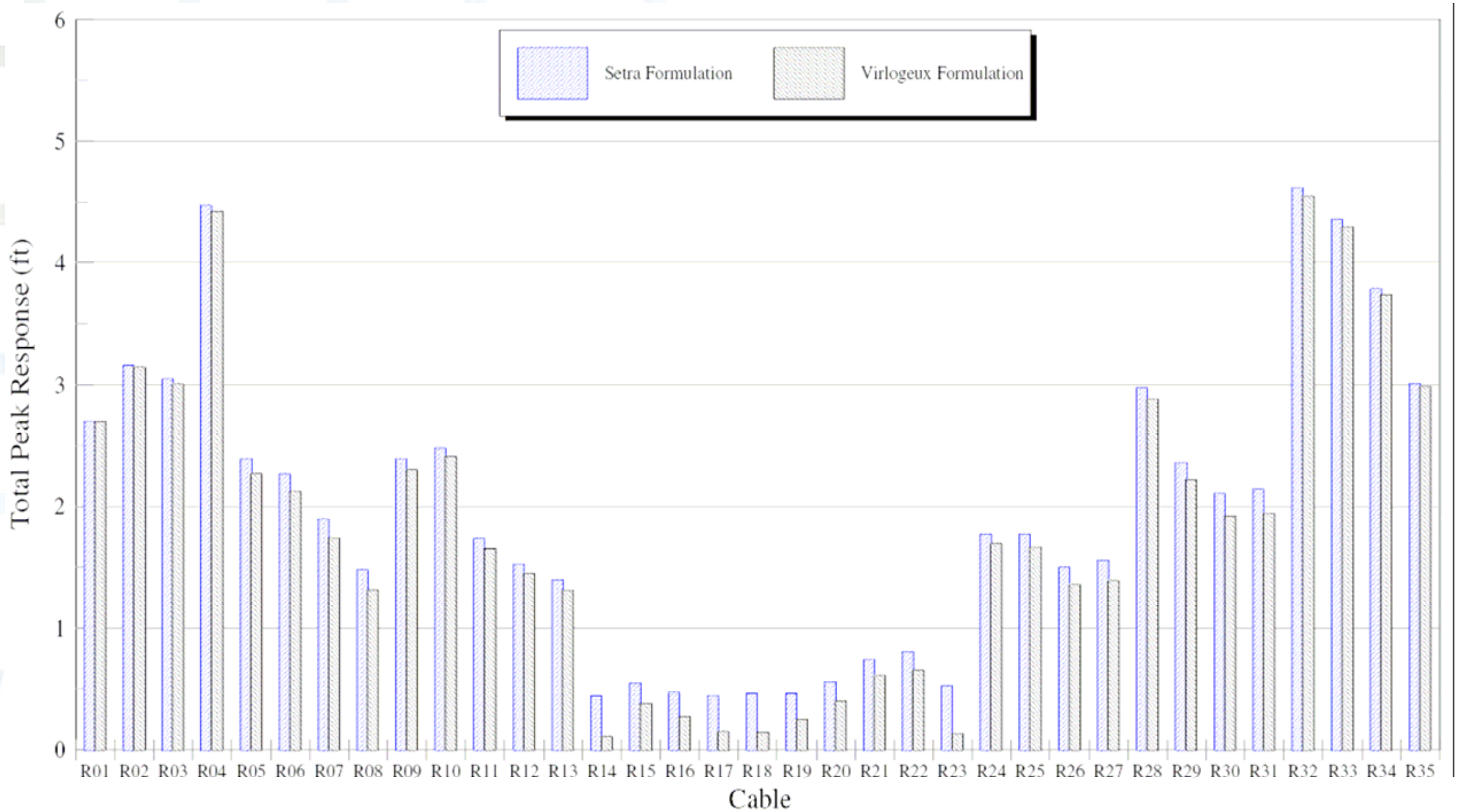
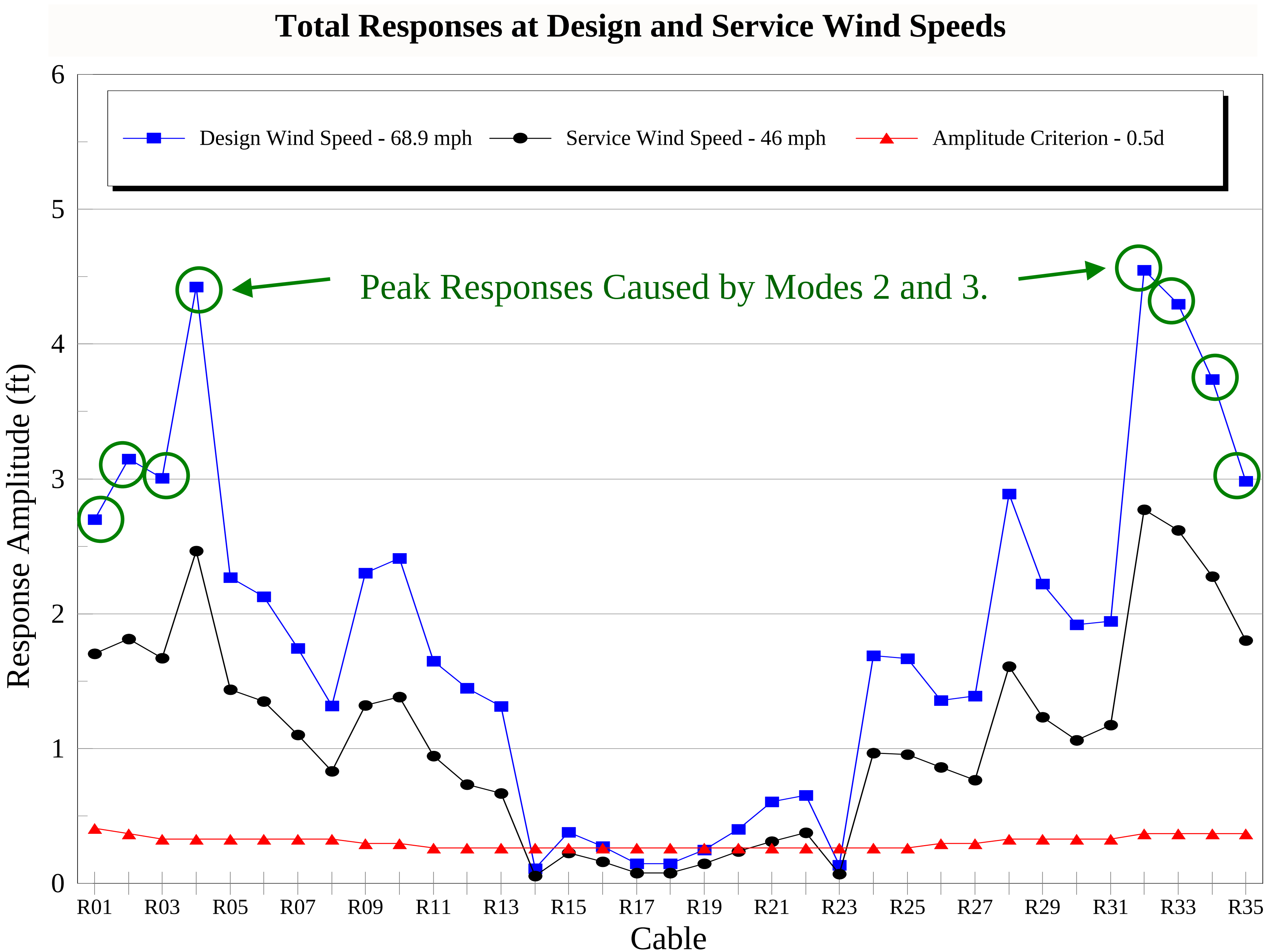


Figure 1: Cable Parametric Response

Design Wind Speed - 68.9 mph



Total Responses at Design and Service Wind Speeds
Design Wind Speed - 68.9 mph
Service Wind Speed - 46 mph
Amplitude Criterion - 0.5d
Peak Responses Caused by Modes 2 and 3.
Response Amplitude (ft)
0
1
2
3
4
5
6
R01
R03
R05
R07
R09
R11
R13
R15
R17
R19
R21
R23
R25
R27
R29
R31
R33
R35
Cable

Assessment Conclusions

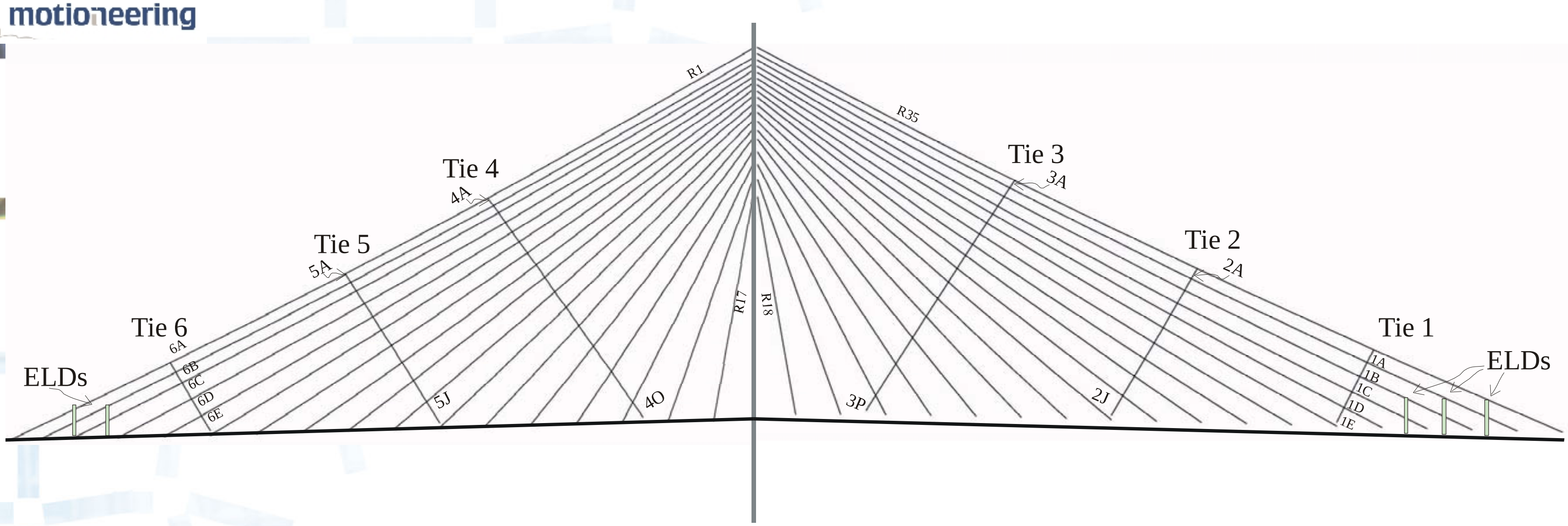
Ironton-Russell Bridge

- Stability control demands for Vortex Shedding, Rain-Wind Vibrations and Dry Galloping lower than the demands for suppressing Motion-Induced Vibrations
- Parametric instabilities – unlikely
- Motion-induced amplitudes would exceed 0.5*D* criterion
- Largest vibration amplitudes associated with mode 2 of the tower and mode 3 of the deck
- Vibration mitigation will be required

Proposed Vibration Suppressing System

Proposed Mitigation Scheme

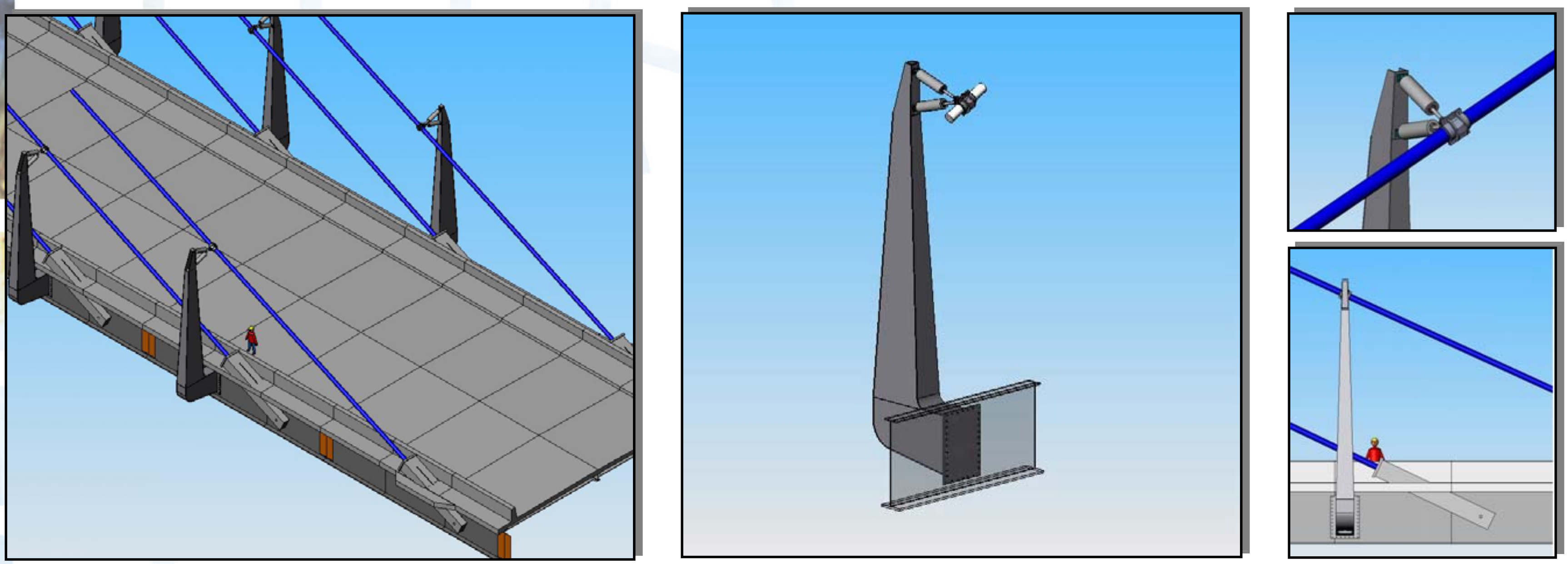
Ironton-Russell Bridge



- Cross-Tie System to control in-plane motions
- External Lateral Dampers (ELDs) to control sway motions.

External Lateral Dampers

Design by Motioneering Inc.

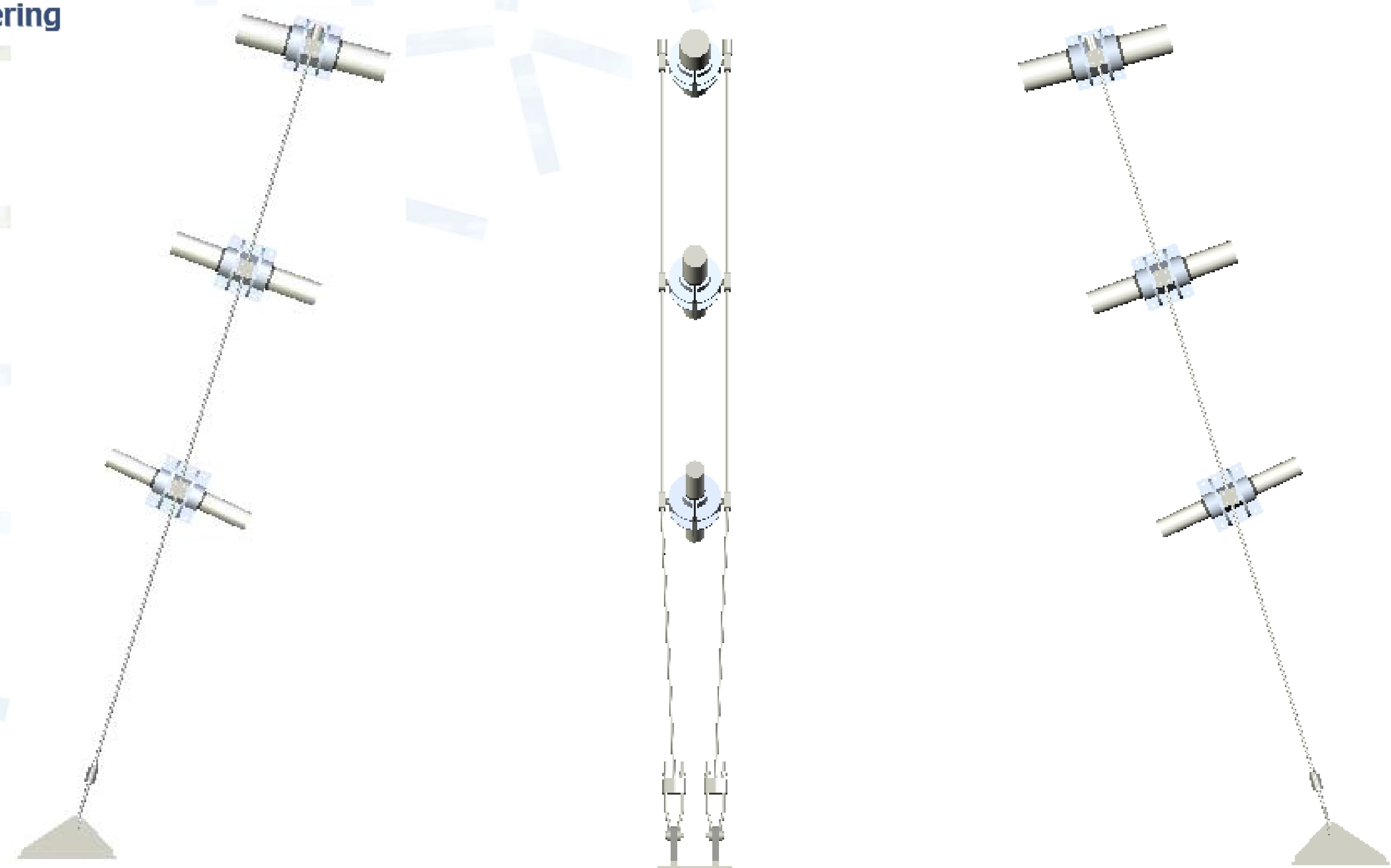


- ELDs will be installed on 10 of the longest cables.
- Expected to contribute approximately 4% modal damping to the cables.

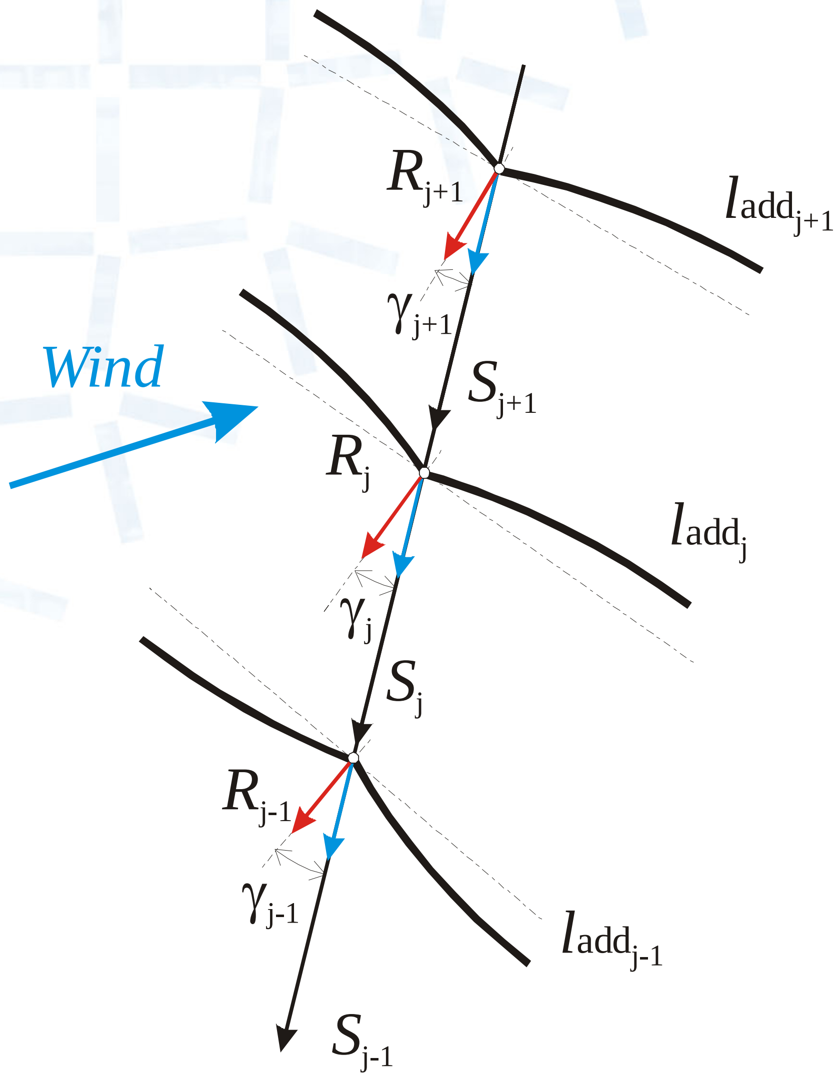
External Lateral Dampers

Design by Motioneering Inc.

Conceptual Design by Genesis Structures



Cross-tie Tension



Accumulation of tensions in a crosstie cable

Estimation of Cross-tie Tension

Loading Scenarios:

1) winds normal to bridge; and

2) winds at skew angles to the along bridge axis

Loads induced either by:

a) a direct buffeting on the cables; or

b) from external excitations such as wind buffeting on the bridge

(motion & parametric excitation will always be present)

Maximum values do not occur for the same wind direction.

For Scenario (1) tie forces due to (a) are minimal and to (b) maximal

For Scenario (2) tie forces due to (a) are maximal and (b) minimal

For this bridge Scenario (2) was found critical

Numerical Modeling of Crossties

Ironton-Russell Bridge

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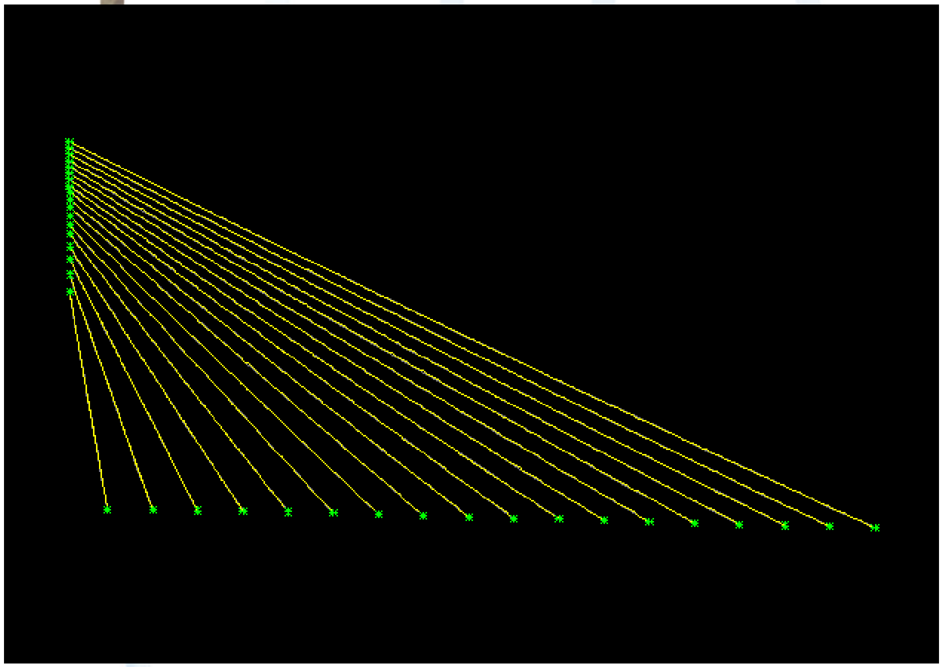
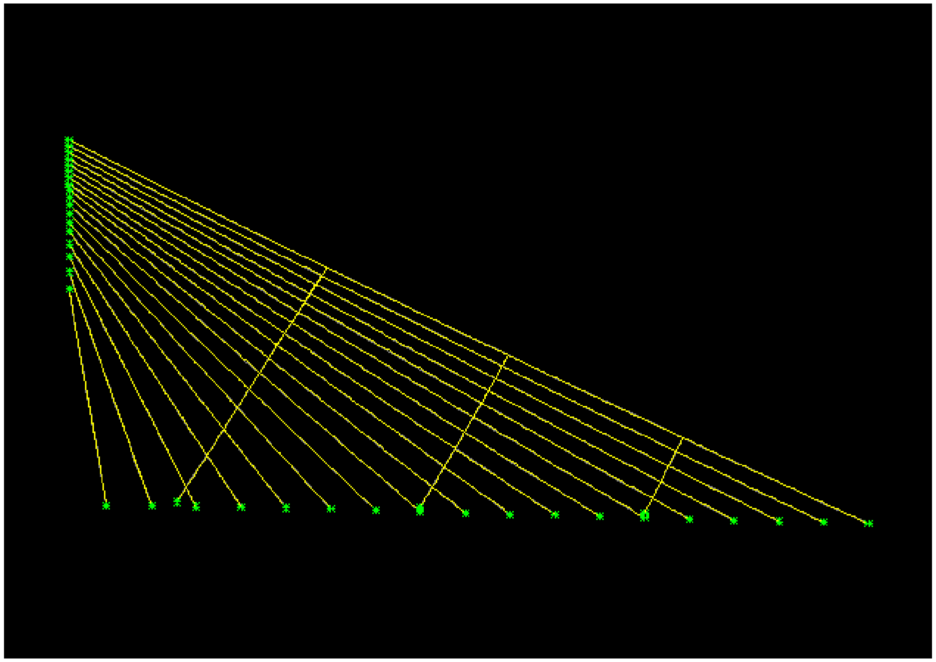
1st Vertical cable mode
0.44 Hz



1st Vertical fan mode
1.12 Hz



SAP2000 nonlinear

Frequencies are de-tuned

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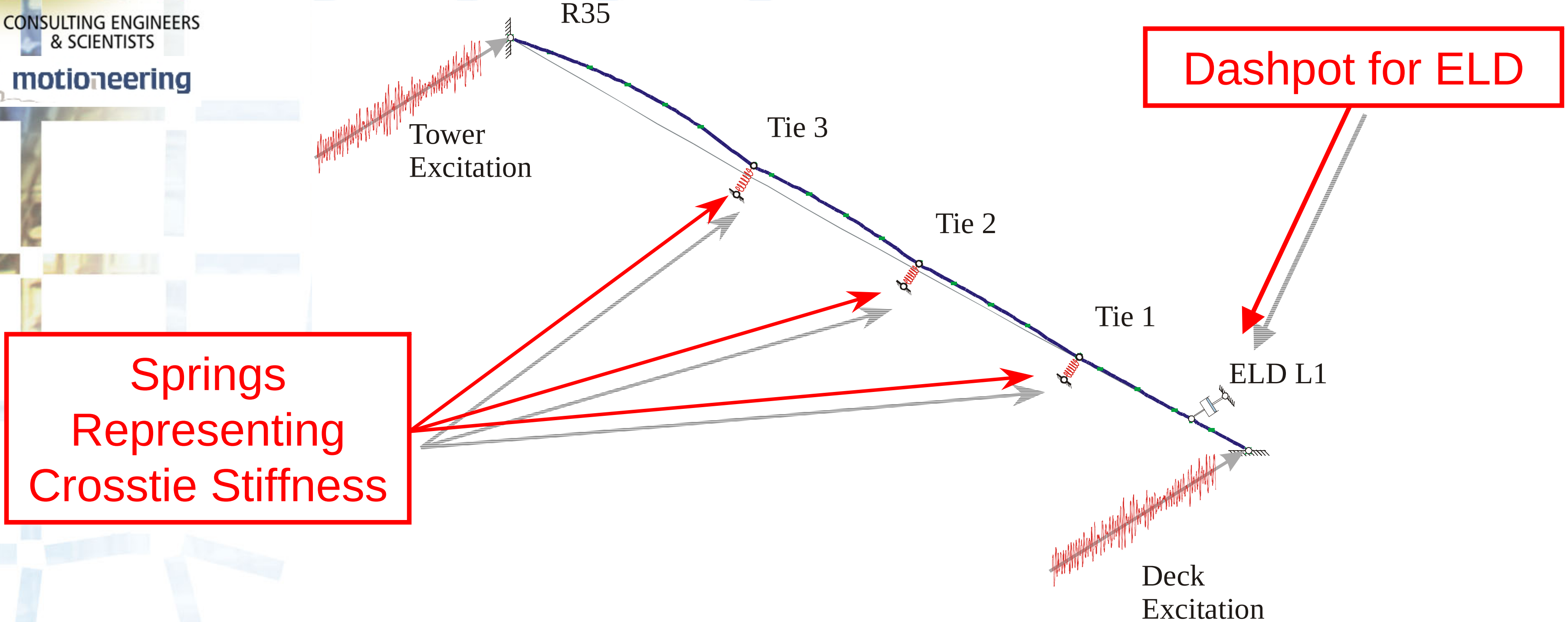
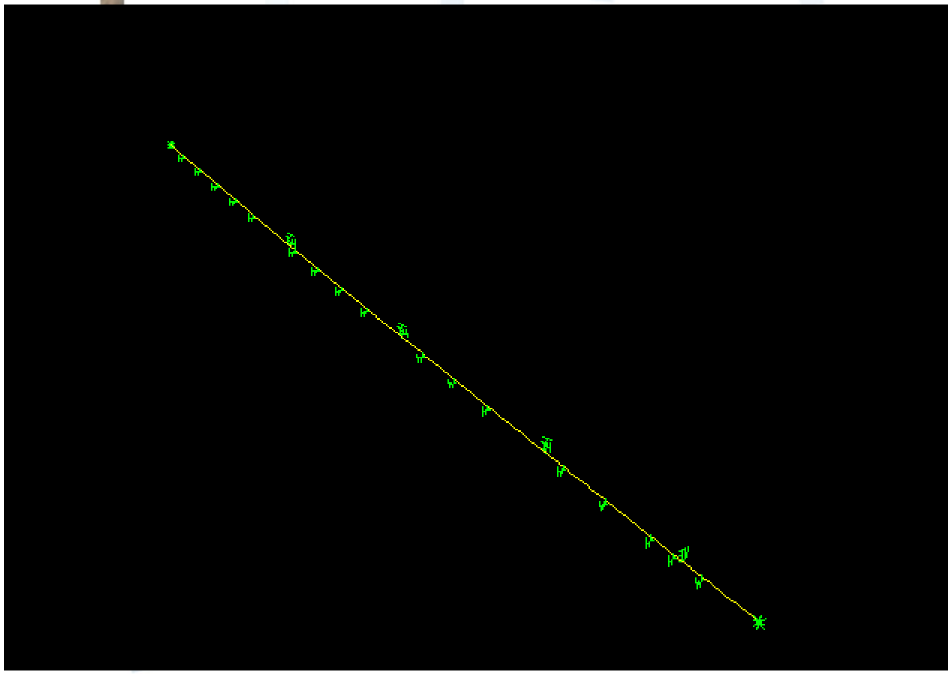
Modeling of Crossties and ELD:
Reduced FEM Cable R35



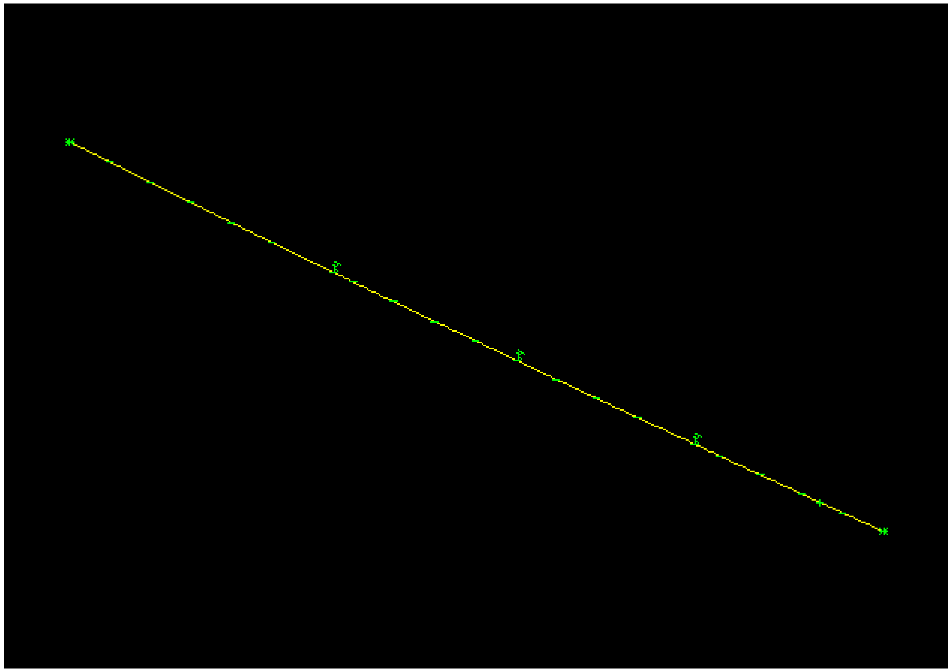
- Reduced FEM of longest cable
- Direct buffeting responses on cables included in the analysis.

Modeling of Crossties and ELD:
Reduced FEM Cable R35

1st Lateral Mode - 0.46 Hz



1st Vertical - 1.11 Hz

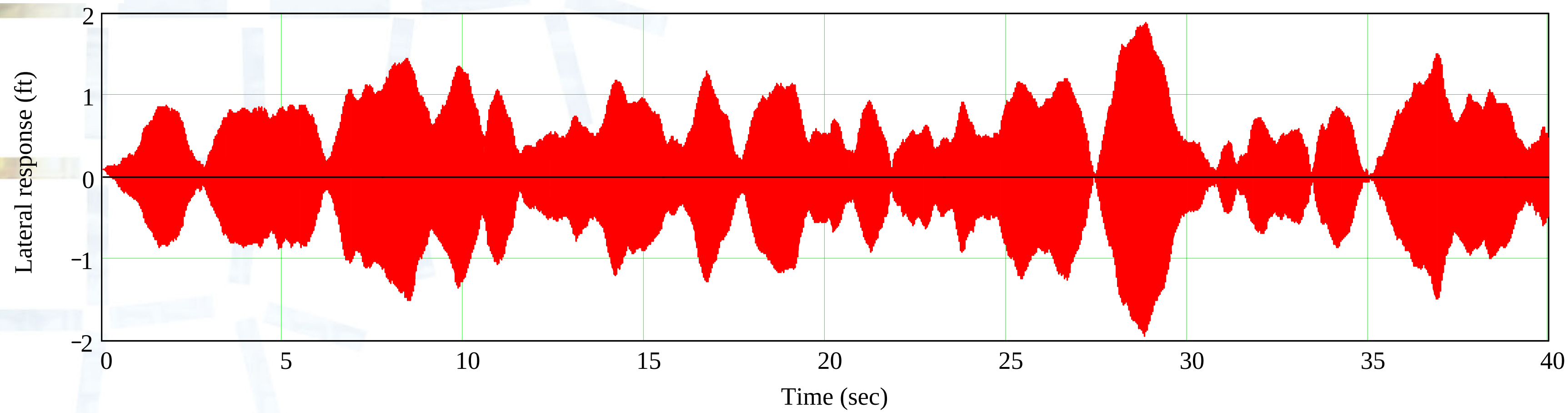


SAP2000 nonlinear

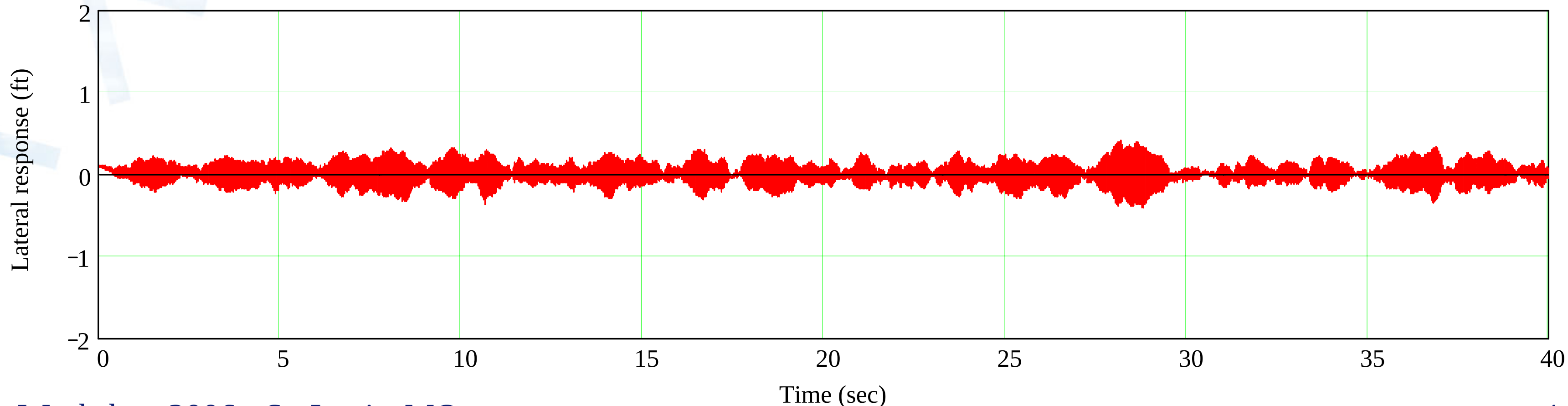
Modeling of Crossties and ELD:

Typical Response Time Histories

R35 Response to buffeting (no mitigation)



R35 Response to buffeting with ELD installed



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Case Study Conclusions

- Due to the tower motions, excessive lateral sway motions of the longest cables were predicted

- Excessive in-plane vibration of several cables also predicted, due to vertical deck motion

- The vibration suppressing system included ELDs and crossties

- The proposed mitigation scheme is expected to reduce
  - to less than 0.5*D* for monthly occurring winds; and
  - to approximately 1*D* during a design windstorm event

General Considerations

- Cable vibration assessment should be applied to all cable-stayed bridges during their design
- This assessment should include all know vibration phenomena VIO, buffeting, galloping, tower-cable-deck interaction for bridge modes that cover the range of possible excitation, e.g. up to 1.5 Hz for wind and up to 4 Hz if pedestrian or vehicle traffic vibrations are of concern
- Analytical motion-induced and parametric excitation analysis methods provide good initial estimates of cable vibrations
- Detailed assessments of displacements prediction could be attained via numerical simulations

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Thank You

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