ENCE717 – Bridge Engineering
Special Topics of Bridges III

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Part III – Special Topics of Bridges

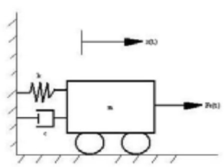
6. Dynamic/Earthquake Analysis (17.0)

i. Basics of Bridge Dynamic Analysis
ii. Vehicle-Bridge Interaction
iii. Pedestrian Bridge Vibrations
iv. Bridge Earthquake Analysis
v. Blast loading Analysis
vi. Wind Analysis

i. Basics of Bridge Dynamic Analysis

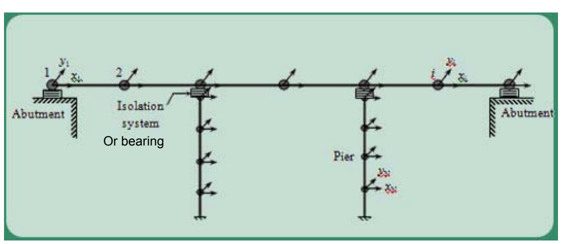
- Single Degree-of-Freedom System

$$m\ddot{y}(t) + c\dot{y}(t) + ky(t) = f(t)$$



- Multiple Degree-of-Freedom System

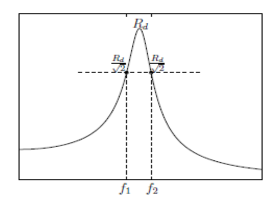
$$\boldsymbol{M}\ddot{\boldsymbol{a}}(t) + \boldsymbol{C}\dot{\boldsymbol{a}}(t) + \boldsymbol{K}\boldsymbol{a}(t) = \boldsymbol{f}(t)$$



i. Basics of Bridge Dynamic Analysis

- Most commonly used experimental method to determine the damping in the structure is the Half-Power (Band-Width) method by two frequencies

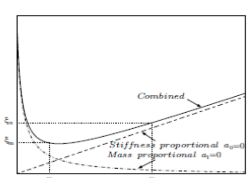
$$\xi = \frac{f_2 - f_1}{f_2 + f_1}$$

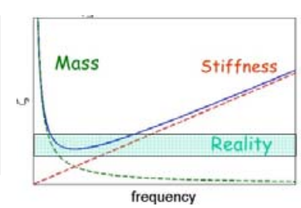


- Mathematically the most common and easy way is to use Rayleigh damping method with a linear combination of the mass and the stiffness matrices

$$c = a_0 m - a_1 k$$

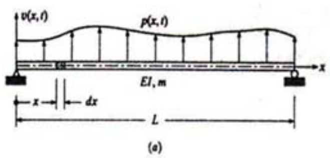
$$\begin{Bmatrix} a_0 \\ a_1 \end{Bmatrix} = \frac{2}{\omega_n + \omega_m} \begin{Bmatrix} \omega_n \omega_m \\ 1 \end{Bmatrix}$$

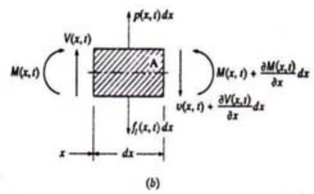


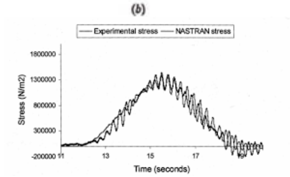


ii. Vehicle-Bridge Interaction

- Aim: To analyze the effects of highway vehicle- or train-induced vibrations for impact analysis or fatigue or cracking analysis.
- In the modeling process, only the superstructure is of a concern to be included in a beam, grid, or more sophisticated shell model.
- The contact force interacting with two substructures, the bridge and the vehicle/train, is time-dependent and nonlinear since the contact force might move from time to time.

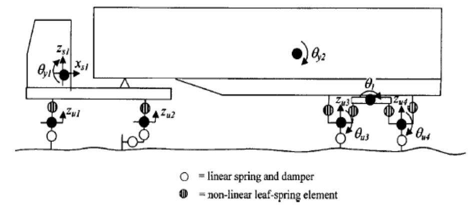


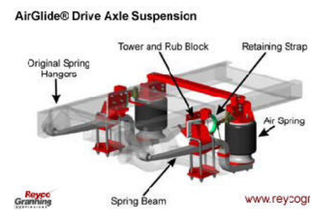


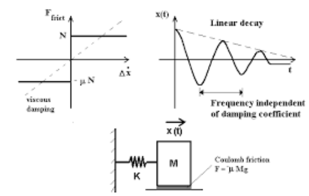


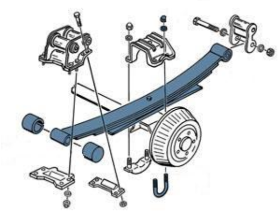
ii. Vehicle-Bridge Interaction

- All vehicles possess the suspension system, either in air suspensions or steel-leaf suspensions. Air suspensions use hydraulic shock absorbers for damping while steel-leaf suspensions use steel strips to provide damping through Coulomb friction between steel strips.



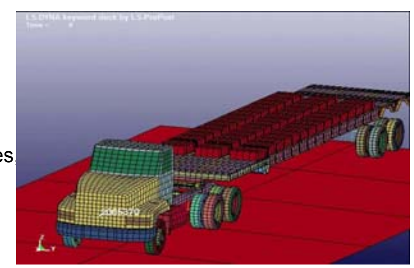


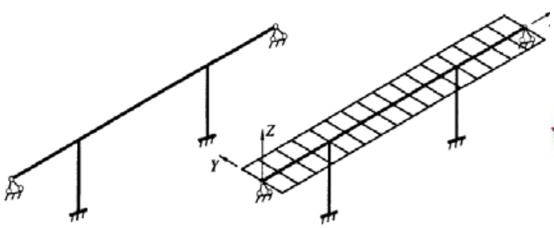


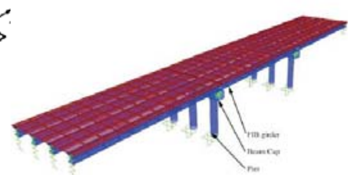


ii. Vehicle-Bridge Interaction

- Bridge can be in beam, grid, or more sophisticated shell model
- Truck can be modeled in details
- Study
  1. dynamic analysis of bridge due to moving vehicles
  2. fatigue life assessment,
  3. quantification bridge durability
  4. heavy vehicle load investigation







iii. Pedestrian Bridge Vibrations

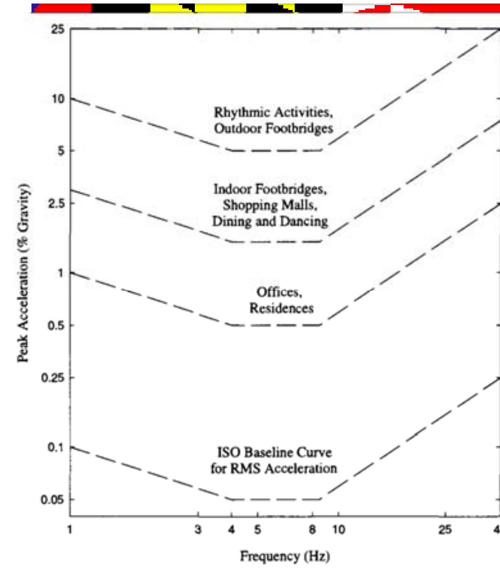


Figure 17.6 - Recommended peak acceleration for human comfort for vibrations due to human activities (Allen and Murray, 1993; AISC 1997)

Case Study: Millennium Bridge

- Crosses River Thames, London, England
- 474’ main span, 266’ north span, 350’ south span

- Superstructure supported by lateral supporting cables (7’ sag)
- Bridge opened June 2000, closed 2 days later

$$\sum f(t) = P\left[1+\sum \alpha_i \cos(2\pi i f_{step} t + \varphi_i)\right]$$

Millennium Bridge

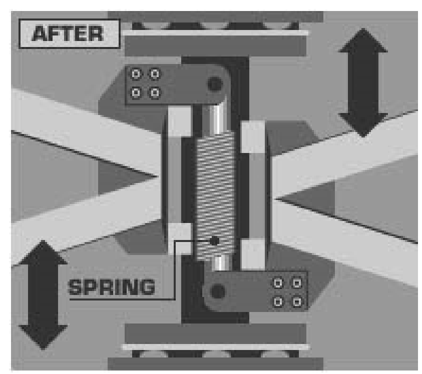
- Severe lateral resonance was noted (0.25g)
- Predominantly noted during 1st mode of south span (0.8 Hz) and 1st and 2nd modes of main span (0.5 Hz and 0.9 Hz)
- Occurred only when heavily congested
- Phenomenon called “Synchronous Lateral Excitation”

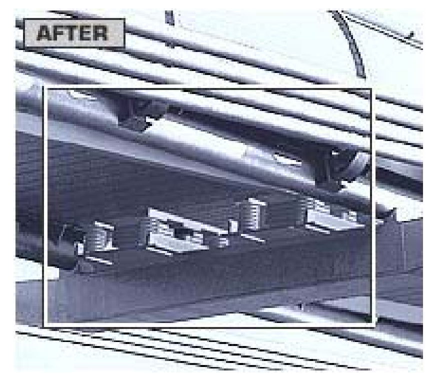
Millennium Bridge

- Possible solutions
  - Stiffen the bridge
    - Too costly
    - Affected aesthetic vision of the bridge
  - Limit pedestrian traffic
    - Not feasible
  - Active damping
    - Complicated
    - Costly
    - Unproven
  - Passive damping

Millennium Bridge

- Passive Dampers
  - 37 viscous dampers installed
  - 19 TMDs installed





Millennium Bridge

- Results
  - Provided 20% critical damping.
  - Bridge was reopened February, 2002.
  - Extensive research leads to eventual updating of design code.

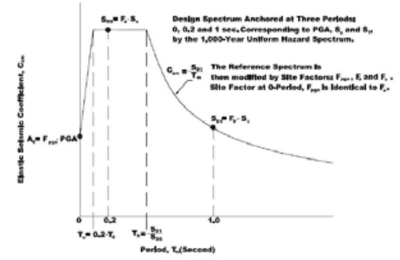
iv. Bridge Earthquake Analysis

TABLE 2-2 PERFORMANCE- BASED SEISMIC DESIGN CRITERIA FOR TRANSPORTATION FACILITIES (NCHRP, 2001)

Probability of Exceedance For Design Earthquake Ground Motions(4)		Performance Level(1)	
		Life Safety	Operation
Rare Earthquake (MCE) 3% PE in 75 years/1.5 mean Deterministic	Service(2)	Significant Disruption	Immediate
	Damage(3)	Significant	Minimal
Expected Earthquake 50% PE in 75 years	Service	Immediate	Immediate
	Damage	Minimal	Minimal to None

SEISMIC DESIGN CATEGORIES AS DEFINED BY AASHTO (2014)

Value of $S_{D1} = F_V S_1$	Seismic Design Category (SDC)
$S_{D1} < 0.15$	A
$0.15 \leq S_{D1} < 0.30$	B
$0.30 \leq S_{D1} < 0.50$	C
$0.50 \leq S_{D1}$	D



Seismic Coefficient Design Spectrum Constructed with the Three-Point Method

iv. Bridge Earthquake Analysis

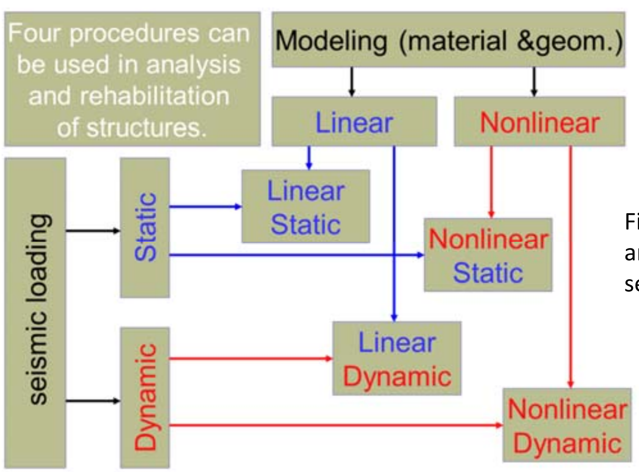
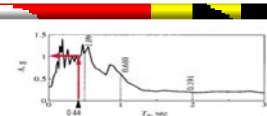
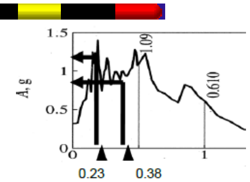


Figure 17.7 - Four distinct analytical procedures for seismic analysis.

AASHTO Guide Specifications in LRFD (2012), differing from the early practices, is adopting displacement-based design procedures instead of the traditional force-based “R-Factor” method.

iv. Bridge Earthquake Analysis





Methods of Analysis -

- Uniform Load Method (single mode, elastic)
- Single Mode Spectral Analysis Method (single mode, elastic)
- Multi Mode Spectral Analysis Method (multiple mode, elastic)
- Elastic Time History (multiple mode, elastic)
- Nonlinear Static Procedure (single DOF, nonlinear)
- Nonlinear Dynamic Procedure (multi DOF, nonlinear)

iv. Bridge Earthquake Analysis

$$m\ddot{u} + c\dot{u} + ku = -m\,\iota\,\ddot{u}_g(t)$$

Table 17,1 – Bridge seismic analysis types recommended by Caltrans

Bridge Classification	Nonlinear Static		Dynamic		
	Equivalent Static Analysis (ESA)	Incremental Static Analysis (Pushover)	Response Spectrum Analysis (RSA)-Linear	Time History Analysis (THA)- Direct integration: Linear	Time History Analysis (THA)- Direct integration: Nonlinear
Ordinary Standard	A	R	A	A	A
Ordinary Nonstandard	N	R	A	A	R
Important	N	R	A	A	R

N: Not acceptable analysis type

A: Acceptable analysis type

R: Acceptable and strongly recommended analysis type, not necessarily comprehensive

iv. Bridge Earthquake Analysis

Probability of Exceedance For Design Earthquake Ground Motions		Performance Level: Life Safety	Performance Level: Operational
Rare Earthquake (MCE) 3% in 75 years	Service	Significant disruption	Immediate
	Damage	Significant	Minimal
Frequent of Expected Earthquake 50% in 75 years	Service	Immediate	Immediate
	Damage	Minimal	Minimal to none

Table 17.2 - Performance Approach

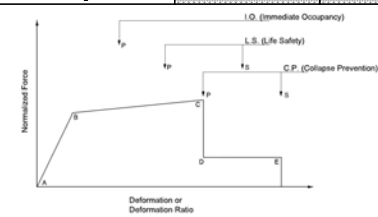
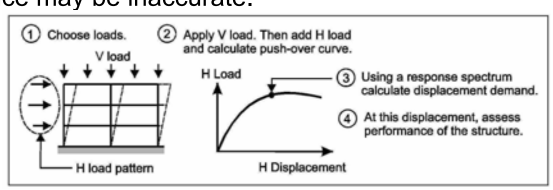


Figure 17.8 - Performance level of structures.

iv. Bridge Earthquake Analysis

Static push-over analysis is an attractive tool for performance assessment because it involves less calculation than nonlinear dynamic analysis, and uses a response spectrum rather than a suite of ground accelerograms. Its main weakness is that it uses static analysis to capture dynamic effects, and hence may be inaccurate.



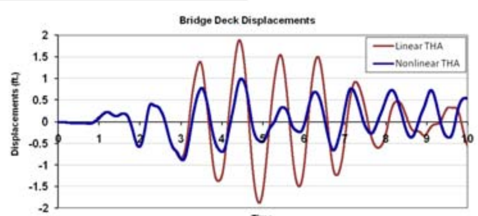


Figure 17.9 - Linear vs. Nonlinear time history analysis for a 9-Span bridge model (THA – Time-History Analysis).

iv. Bridge Earthquake Analysis

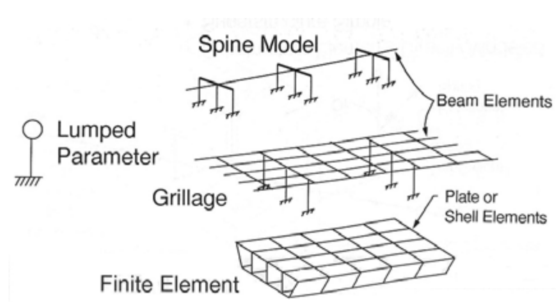


Figure 17.11 – Types of Analytical Models

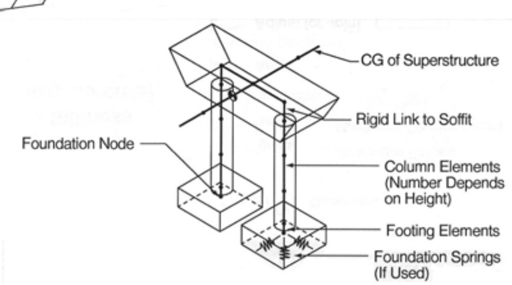
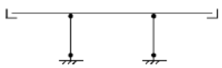
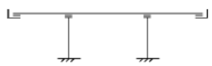
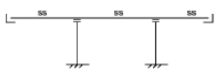
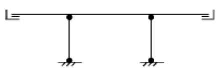
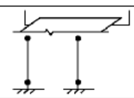
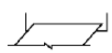


Figure 17.12 – Illustration of a spine model

iv. Bridge Earthquake Analysis

TABLE 4-4 PERMISSIBLE EARTHQUAKE RESISTING SYSTEMS

Case	Earthquake Resisting Systems	Description of ERS
1		Longitudinal Response: • Plastic hinges in inspectable locations or elastic design of columns. • Abutment resistance not requires as part of ERS. • Knock-off backwalls of abutments permissible.
2		Longitudinal Response: • Isolation bearings accommodate full displacement. • Abutment resistance not requires as part of ERS.
3		Longitudinal Response: • Multiple simply supported spans with adequate support lengths. • Plastic hinges in inspectable locations or elastic design of columns.
4		Longitudinal or Transverse Response: • Plastic hinges in inspectable locations or elastic design of columns. • Isolation bearings with or without energy dissipaters to limit overall displacements.
5		Transverse Response: • Plastic hinges in inspectable locations or elastic design of columns. • Abutment not required in ERS, breakaway shear keys permissible.
6		Transverse or Longitudinal Response: • Abutment required to resist design earthquake elastically. • Longitudinal passive pressure less than 70 percent of maximum that can be mobilized.

iv. Bridge Earthquake Analysis

Table 17.5 – Linear and Nonlinear Component Modeling

Component	Linear-Elastic	Nonlinear
Superstructure	X	
Column–plastic hinge zone		X
Column–outside plastic hinge zone	X	
Cap beam	X	
Abutment– transverse		X
Abutment– longitudinal		X
Abutment– overturning		X
Abutment– gap		X
Expansion joints		X
Foundation springs	X	
Soil-structure interaction	X	

iv. Bridge Earthquake Analysis

- The **superstructure** is idealized using equivalent linear elastic beam-column elements
- Effective bending stiffness - the moment of inertia I_{eff}

$$E_c I_{eff} = \frac{M_y}{\varphi_y} \qquad (17.18)$$

- Shear stiffness parameter $(GA)_{eff}$ for pier walls in the strong direction

$$(GA)_{eff} = G_c A_{cw} \frac{I_{eff}}{I_g} \qquad (17.19)$$

- Effective torsional moment of inertia J_{eff}

$$J_{eff} = 0.2 J_g \qquad (17.20)$$

iv. Bridge Earthquake Analysis

- **Soil Stiffness**: Abutment longitudinal stiffness K_{eff} due to passive soil pressure uniformly distributed over the height (H_w) and width (W_w) of the back-wall or diaphragm.

$$P_p = p_p H_w W_w \qquad (17\text{-}21)$$

- For integral- or diaphragm-type abutments, equivalent linear secant stiffness, K_{eff} is

$$K_{eff} = \frac{P_p}{(F_w H_w)} \qquad (17\text{-}22)$$

iv. Bridge Earthquake Analysis

Table 17.4 – Stiffness of Circular Surface Footing (K_0)

Degree of Freedom	Equivalent Radius R	Stiffness K_0
Vertical Translation	$R_0 = \sqrt{\frac{4BL}{\pi}}$	$4GR/(1-\nu)$
Lateral Translation (Both)	"	$8GR/(2-\nu)$
Torsion Rotation	$R_1 = \left[\frac{4BL(4B^2+4L^2)}{6\pi}\right]^{1/4}$	$16GR^3/3$
Rocking about 2	$R_2 = \left[\frac{(2B)^3(2L)}{3\pi}\right]^{1/4}$	$8GR^3/3(1-\nu)$
Rocking about 3	$R_3 = \left[\frac{(2B)(2L)^3}{3\pi}\right]^{1/4}$	"

iv. Bridge Earthquake Analysis

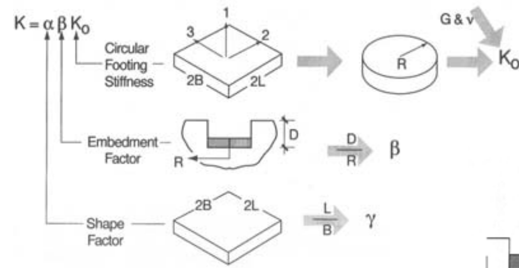


Figure 17.13 – Half-spaced method for spread footings (NHI 1996)

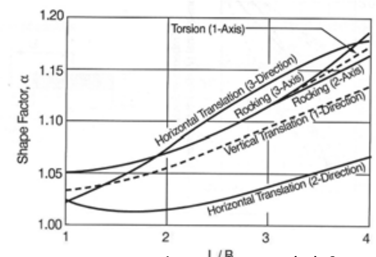


Figure 17.14 – Shape factor (α) for rectangular footing (NHI 1996)

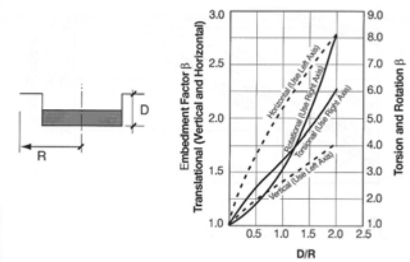


Figure 17.15 – Embedment factor (β)

iv. Bridge Earthquake Analysis

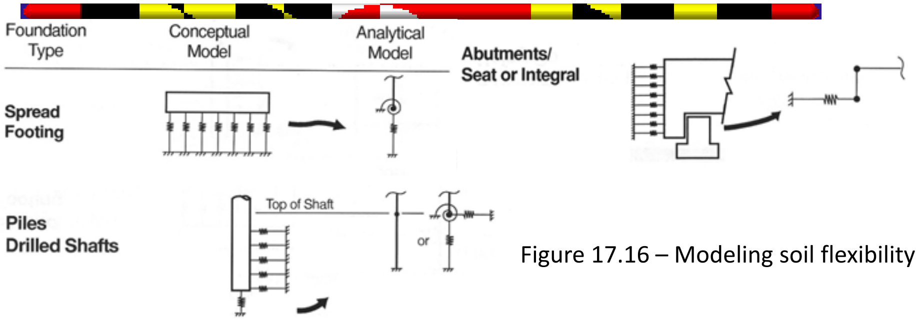


Figure 17.16 – Modeling soil flexibility

Foundation Type	Modeling Method I	Modeling Method II
Spread Footing	Rigid	Foundation spring required if footing flexibility contributes more than 20% to pier displacement
Pile Footing with Pile Cap	Rigid	Foundation spring required if footing flexibility contributes more than 20% to pier displacement
Pile Bent/Drilled Shaft	Estimate depth to fixity	Estimate depth to fixity or soil springs based on P-y curves

iv. Bridge Earthquake Analysis

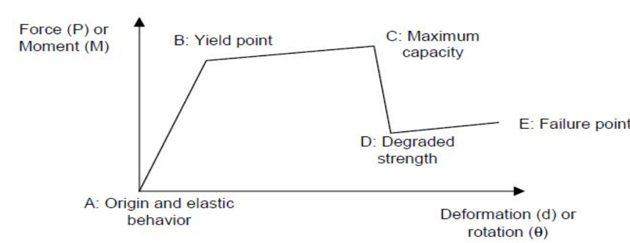
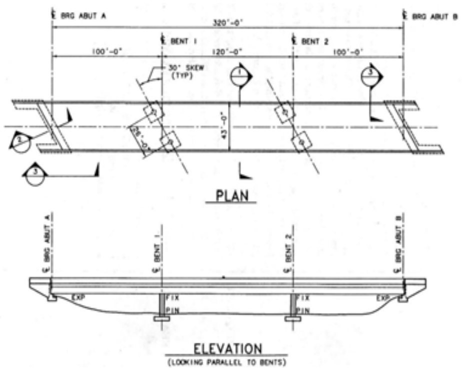


Figure 17.17 – Pushover force-deformation (P-d) or moment rotation (M-Θ) curve

plastic rotation capacity angle, *a from B to C*
ultimate rotation angle, *b* from B to E (i.5 times the plastic angle)
Modal Pushover Analysis (MPA) - pushover analyses are carried our separately for each significant mode, and the contributions from individual modes to calculated response quantities (displacements, drifts, etc.) are combined using an appropriate combination rule (SRSS or CQC).

iv. Bridge Earthquake Analysis

FHWA Example



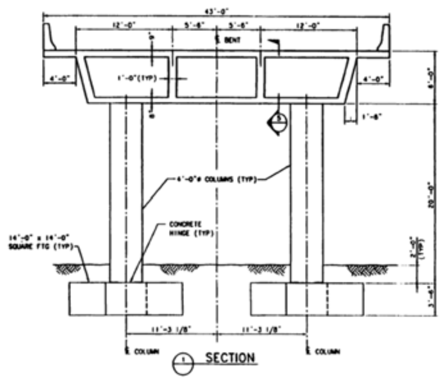


Figure 17.18 – Plan and elevation views of illustration example 1 (FHWA 1996)

iv. Bridge Earthquake Analysis

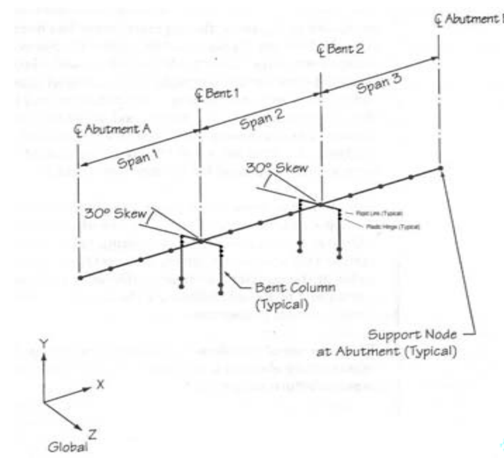


Figure 17.20 - Finite element model of illustration example 1 (FHWA 1996)

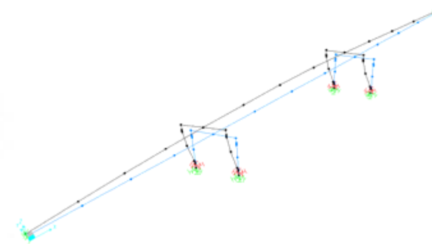


Figure 17.22 – Deformed shape of Mode 2 (T_2 = 0.5621s)

iv. Bridge Earthquake Analysis

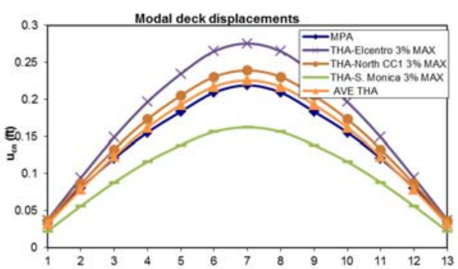
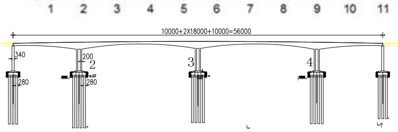
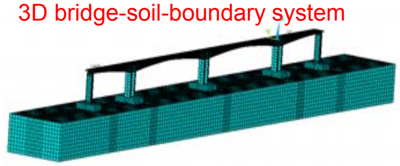


Figure 17.23 – Comparison of different methods by deck displacement



3D bridge-soil-boundary system



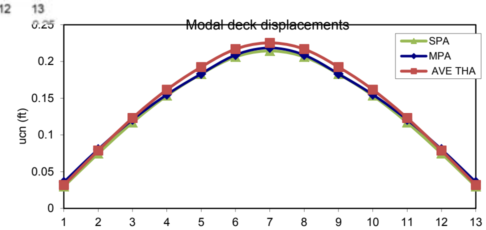


Figure 17.24 – Comparison of different methods by deck displacement (PGA = 0.30g)

v. Blast loading Analysis

Analysis for blast-resistant design:

1) **Equivalent static analysis** (neglecting the inertial effects of members in motion)
2) **Single-degree-of-freedom (SDOF) linear/nonlinear dynamic analysis** (considered the current state-of- practice method which ignores higher-order failure, allowing for the analysis of a large number of load cases, bridge types, and structural configurations)
3) **Multi-degree-of-freedom (MDOF), uncoupled/ coupled, nonlinear dynamic analysis**

Modified Friedlander exponential decay equation

$$p(t) = p_m[1 - t/t_p] e^{-\alpha t/t_p}$$

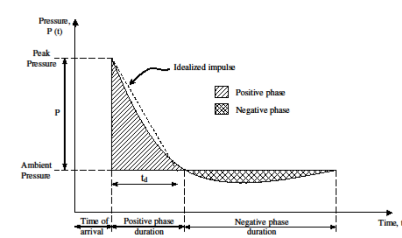
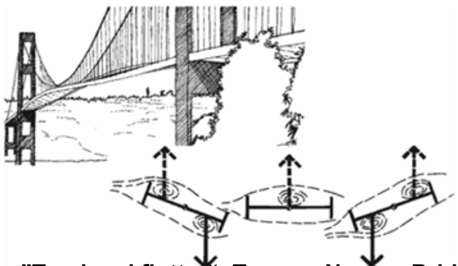


Figure 17.10 – Pressure time-history for free field blast (TM5-1300 1990)

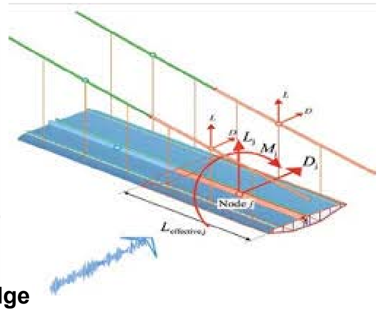
vi. Wind Analysis

Wind induces two typical aerodynamic phenomena in long span bridges:

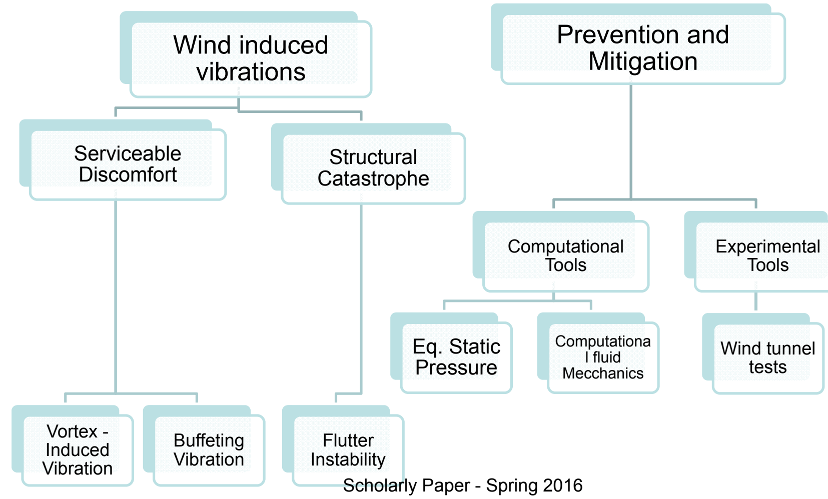
- **Fluttering** is an aerodynamic instability that may cause failure of the bridge
- **Buffeting** is an aerodynamic random vibration that may lead to fatigue damage, excessive vibration, and large displacements.



"Torsional flutter": Tacoma Narrow Bridge



vi. Wind Analysis



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vi. Wind Analysis

- Aerodynamic loading is commonly separated into self-excited and buffeting forces.
- The self-excited forces acting on a unit deck length are expressed as a function of the so-called flutter derivatives (Scanlan 1978a), which can be expressed as:
- $$\{F_{se}\} = \begin{Bmatrix} L_{se} \\ D_{se} \\ M_{se} \end{Bmatrix} = U^2[F_d]\{q\} + U^2[F_v]\{q\} \qquad (17.16)$$
- The buffeting forces (Scanlan 1978b) are expressed as

$$\{F_b\} = \begin{Bmatrix} L_b \\ D_b \\ M_b \end{Bmatrix} = \bar{U}^2[C_b]\{\eta\}$$

vi. Wind Analysis

- Wind is a dynamic load. However, it is generally approximated as a uniformly distributed static load on the exposed area of a bridge.
- For typical girder and slab bridges (based on 100 mph)
  - SPAN ≥ 125': 0.05 ksf, transverse, 0.02 ksf, longitudinal
  - SPAN < 125': 0.10 ksf, transverse, 0.04 ksf, longitudinal
- For the strength limit state, wind on the structure is considered for the Strength III and Strength V load combinations. For Strength III, the load factor for wind on structure is 1.40 but live load is not considered. Therefore, for this design example, only the Strength V load combination will be investigated. The Strength III load combination is likely to be more critical when checking wind load effects during construction.

STRENGTH-III	γ_P	-	1.00	1.40	
STRENGTH-V	γ_P	1.35	1.00	0.40	1.0