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Column Design: Past, Present, Future
December 6, 2018



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AISC Live Webinars

Course Description

Column Design: Past, Present, Future
December 6, 2018

The historical development of the different approaches for designing metal columns is presented and critiqued. Prior to 1960, emphasis was placed on design methods for an isolated compression member. Since 1960 the focus has been on columns as part of frames, by introducing factors such as effective length factors (K-factors), $P\Delta$, frame stability, plastic design and second-order structural analysis. The development of the current AISC column curve is presented. The single-column curve vs the multiple-column curve controversy is evaluated, and possible future changes to our design approach, due to the current methods of manufacturing rolled steel sections, will be predicted.



AISC Live Webinars

Learning Objectives

- Describe how builders proportioned columns historically, from ancient times through the end of the 18th century.
- List the research advances that took place in the 19th century, which informed the design practices for wrought iron and early steel columns.
- Explain the strategies that engineers have employed to account for the effect of residual stresses in rolled steel shapes, in compression members.
- Identify why current manufacturing processes might cause reformulation of structural steel column design methods in the future.



Column Design: Past, Present, Future



Joseph A. Yura, PE, PhD
Emeritus Professor in Civil Engineering
University of Texas at Austin



INTRODUCTION

HISTORICAL REVIEW OF PAST WORK

- Axially Loaded Column Design:
Ancient → 1960
- Column Design as Part of a Frame:
1960 → Present

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INTRODUCTION

HISTORICAL REVIEW OF PAST WORK

What is the purpose of a historic review?

- The best way to teach a subject is in the order it evolved
- Helps guide and predict the future approaches

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OUTLINE: (KEY WORDS)

1. ANCIENT – 1650 (Greek temples, calculus)
2. 1650 -1800 (Euler)
3. 1800 -1900 (wrought iron, eccentrically-loaded column)
4. 1900 -1945 (steel frame, AISC)
5. 1945 -1970 (K-factors, plastic design, residual stress)
6. 1970 -1985 (frames, multiple column curves, LRFD)
7. 1985 -2018 (K = 1.0, 2nd order modified analysis)
8. Future

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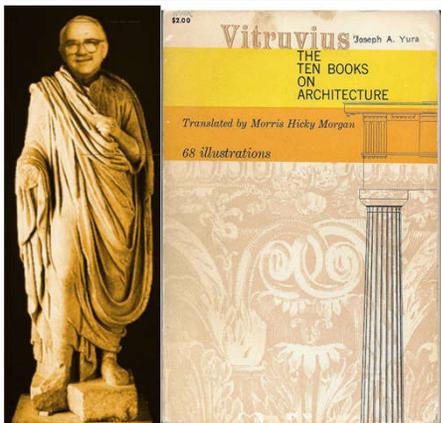
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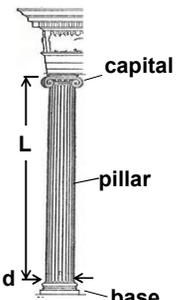
12



5000 BC: THE FIRST COLUMN FORMULA

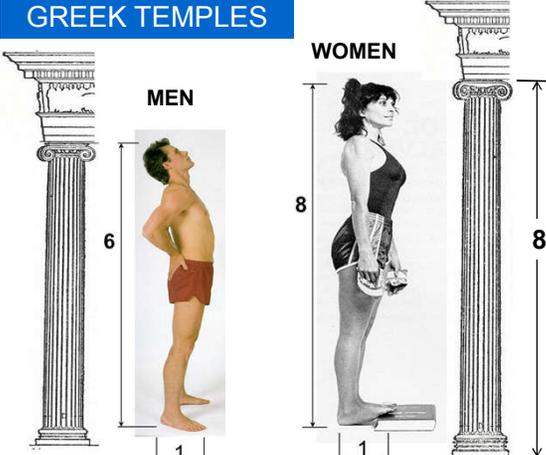


Vitruvius Joseph A. Yura
THE TEN BOOKS ON ARCHITECTURE
Translated by Morris Hicky Morgan
68 illustrations

$$\frac{L}{d} = \frac{\text{Body Height}}{\text{Foot Imprint}}$$


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GREEK TEMPLES



MEN
6
1

WOMEN
8
1

$8 + \text{base} = 10$

$L/d \leq 10$ in ACI Code until 1956

2005 DATA
Men: **6.7**
Women: **7.0**

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ANCIENT - 1650

STRUCTURAL MECHANICS, MATERIALS, MATH

- Concept of equilibrium was understood but stress and strain were unknown
- Stone, masonry and wood columns
- **Trigonometry** fully developed by the 10th century.
Differential calculus developed in 1629

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1650–1800: MECHANICS, MATERIALS, MATH

- Hooke's Law (1660) – $F \propto \Delta L$
John, James Bernoulli (1691) - $d\theta/ds = 1/r \propto M$
- Mainly stone, masonry and wood columns;
some cast iron
- Major math advances
- Straight wooden slide rule

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1650–1800: COLUMN DESIGN

- The earliest cast iron columns just followed the proportions of stone columns
- Two Euler solutions

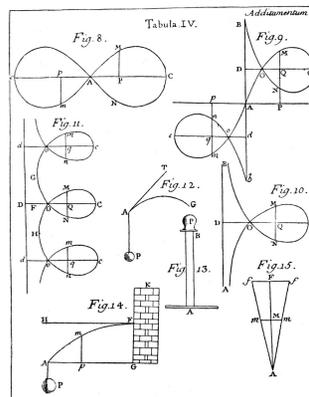
18

EULER (1744). Elastic Curves

Large Deflection Theory

$$\frac{d\theta}{ds} = -\frac{Py}{EI}$$

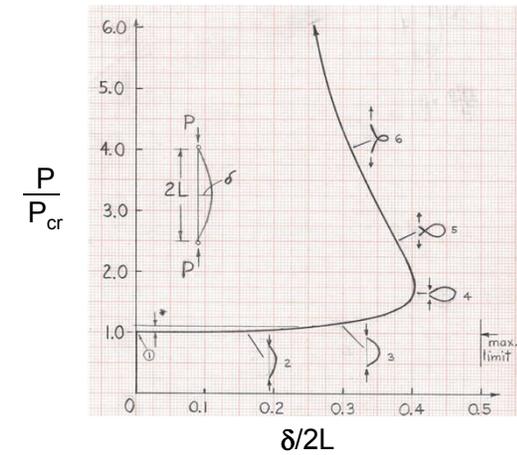
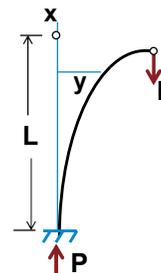
unknown geometric and elastic material term



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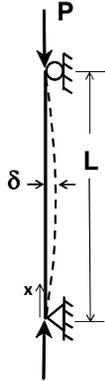
EULER (1744)

$$P_{cr} = \frac{\pi^2 EI}{(2L)^2}$$



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EULER (1757). On the Strength of Columns

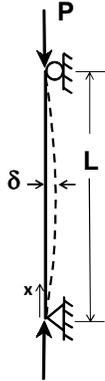


Small deflection theory:

$$y'' = -\frac{Py}{EI} \quad \text{yields} \quad P_E = \frac{\pi^2 EI}{(L)^2}$$

21

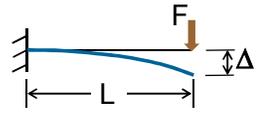
EULER (1757). On the Strength of Columns



Small deflection theory:

$$y'' = -\frac{Py}{EI} \quad \text{yields} \quad P_E = \frac{\pi^2 EI}{(L)^2}$$

Determine EI experimentally:
Measure F , Δ , L -



Solve for EI from

$$\Delta = \frac{FL^3}{3EI}$$

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1800–1880: MECHANICS, MATERIALS, PRACTICE

- **Concepts of stress, strain, E and I established, 1820**
- **Stresses and deflections of simple beams could be calculated**
- **Wrought iron compression members until 1880**
- **Method of joints for truss analysis (Whipple, 1847)**

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TREDGOLD (1822): FIRST COLUMN DESIGN FORMULA

1st YIELD LIMIT $f_a + f_b = F_y$
Assume same deflected shape (error)
 $M_{max} = P(e + \delta)$

Rectangular section

$$\frac{P}{P_y} = \frac{1}{1 + \frac{6e}{d} + \frac{3F_y}{2E} \left(\frac{L}{d}\right)^2}$$

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1800–1880: TEST MACHINES, COLUMN TESTS

- Test Machines and strain instrumentation came available to provide reliable material properties
- Hodgkinson (1840) - 250 column tests
Cast iron, wrought iron and wood
Flat and rounded ends, poorly defined
Euler not verified : $P_{max} \propto 1/L^{1.7}$
- One million lb test machine installed in US in 1879

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SCHEFFLER (1858): EXACT 2ND ORDER ELASTIC ANALYSIS

Secant Formula

1st YIELD LIMIT $f_a + f_b = F_y$

$$\frac{P}{P_y} = \frac{1}{1 + \frac{ed}{2r^2} \sec \left[\left(\frac{L}{2r}\right) \sqrt{\left(\frac{P}{P_y}\right) \left(\frac{F_y}{E}\right)} \right]}$$

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GORDON-RANKINE COLUMN FORMULA (1845, 1858)

Gordon (1845) (altered Tredgold) $\frac{P}{P_y} = \frac{1}{1 + \frac{6e}{d} + \frac{3F_y}{2E} \left(\frac{L}{d}\right)^2}$

fit to Hodgkinson results $\rightarrow a$

Rankine (1845) $\rightarrow r$

Gordon- Rankine $\frac{P}{P_y} = \frac{1}{1 + a \left(\frac{L}{r}\right)^2}$

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GORDON-RANKINE FORMULA (1845, 1858)

Gordon- Rankine
$$\frac{P}{P_y} = \frac{1}{1 + a \left(\frac{L}{r}\right)^2}$$

Wrought Iron Columns: use a FS = 4 for working stress
a= 1/3000 (fixed ends), =4/3000 (pinned ends)

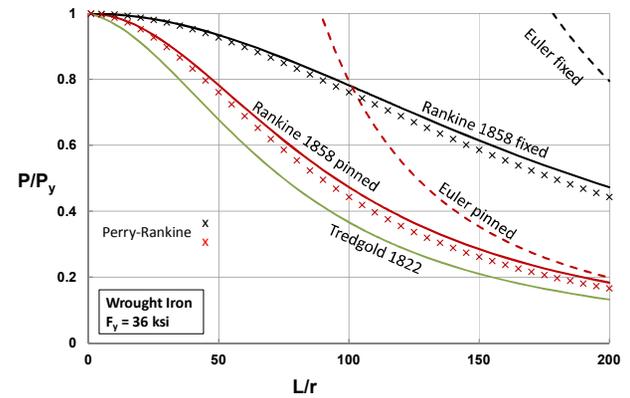
GORDON-RANKINE- PERRY FORMULAS

Gordon- Rankine
$$\frac{P}{P_y} = \frac{1}{1 + a \left(\frac{L}{r}\right)^2}$$

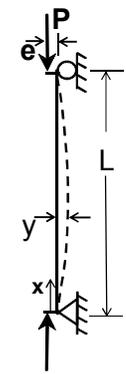
Perry (1889): if $a = \frac{F_y}{(\pi^2 EK^2)}$ 3/2 (Tredgold)

$$\frac{P}{P_y} = \frac{1}{1 + \frac{P_y}{P_{cr}}} \quad \text{or} \quad \frac{1}{P} = \frac{1}{P_y} + \frac{1}{P_{cr}}$$

RANKINE COLUMN CURVES



SCHEFFLER (1858): SECANT FORMULA



1st YIELD LIMIT $f_a + f_b = F_y$

$$M_{max} = P(e + y_{max}) = (Pe) \sec \frac{\pi}{2} \sqrt{\frac{P}{P_E}}$$

$$\frac{P}{P_y} = \frac{1}{1 + \frac{ec}{r^2} \sec \left[\left(\frac{L}{2r}\right) \sqrt{\left(\frac{P}{P_y}\right) \frac{F_y}{E}} \right]}$$

Not a practical column formula
no EXCEL Solver available!!



AYRTON-PERRY (1886) EXACT 2ND ORDER ANALYSIS

Initial Out-of-Straightness, δ_o , (sine curve)

load eccentricity (parabola)

out-of-straightness (sine curve)

Initial Displacement

$e = \delta_o$

$$P\delta = \frac{P\delta_o}{1 - \frac{P}{P_e}}$$

$$f_a + f_b = F_y$$

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AYRTON-PERRY (1886) COLUMN FORMULA

Initial Out-of-Straightness, δ_o , (sine curve)

$$f_a + f_b = F_y$$

$$\frac{P}{P_y} = \frac{1 + (1 + m) \frac{P_e}{P_y}}{2} - \sqrt{\left[\frac{1 + (1 + m) \frac{P_e}{P_y}}{2} \right]^2 - \frac{P_e}{P_y}}$$

where $m = \frac{\delta_o c}{r^2}$

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SECANT AND AYRTON-PERRY 1ST YIELD SOLUTIONS

P/P_y

L/r

Euler

δ_o

e

$x-x$

$y-y$

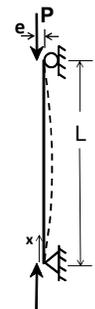
$\delta_o = e = L/1000$
 W12x65, $F_y = 50$ ksi

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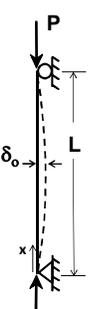
SLIDE RULE Until 1970

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SECANT AND AYRTON-PERRY 1ST YIELD SOLUTIONS



SECANT



PERRY

- Both require an input of an imperfection
- Both were considered too complicated
- Perry could be used to solve the end eccentricity problem by using $\delta_0 = (1.2)e$

USE RANKINE

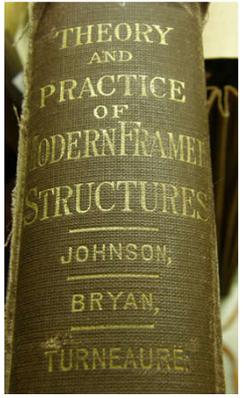
37

1880-1900: MECHANICS, MATERIALS, PRACTICE

- Euler is verified by careful elastic tests with knife - type end fixtures
- Engesser extends Euler theory to inelastic range with two inelastic theories
- Steel starts to replace wrought iron columns
- Railroad bridges failed at the rate of 1/week over a ten-year period
- Engineers developed their own design specs and column formulas.

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FIRST STEEL DESIGN TEXT



THE THEORY AND PRACTICE
OF
MODERN FRAMED STRUCTURES.

DESIGNED FOR THE USE OF SCHOOLS,
AND FOR
ENGINEERS IN PROFESSIONAL PRACTICE.

BY
J. B. JOHNSON, C.E.,
Professor of Civil Engineering in Worcester Polytechnic Institute, Worcester, Mass.; Member of the Institution of Civil Engineers,
Member of the American Society of Civil Engineers, Member of the American Society of Mechanical Engineers,
(1886, 1891)

C. W. BRYAN, C.E.,
Assistant of the Chief-Engineer, Chicago, (Ill.),
(1886)

AND
F. E. TURNEAURE, C.E.,
Professor of Bridge and Structural Engineering, University of Wisconsin, Madison,
(1886)

THIRD EDITION, REVISED,
REVISED THIRDEDITION.

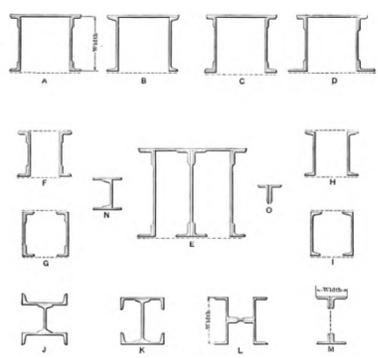
NEW YORK:
JOHN WILEY & SONS,
53 Broadway, N.Y.

1894.

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1800-1900: TYPICAL TRUSS BRIDGE MEMBERS

Johnson et al (1894)



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JOHNSON PARABOLA (1894)

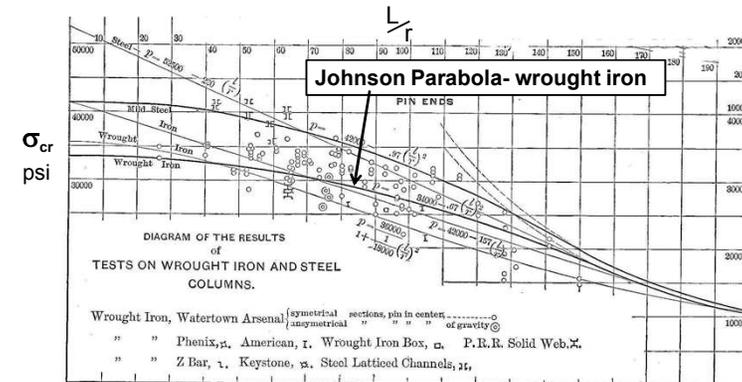
CURVE FIT TO TESTS $\frac{P}{P_y} = 1 - \frac{F_y}{4\pi^2 E} \left(\frac{KL}{r}\right)^2$ for $\frac{P}{P_y} \geq 0.5$;

EULER THEORY $\frac{P}{P_y} = \frac{\pi^2 E}{F_y} \left(\frac{KL}{r}\right)^2$ for $\frac{P}{P_y} < 0.5$

This formulation was adopted in the AISC Spec (1961-1985)

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WROUGHT IRON TESTS (1894)



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1800–1900: ENGINEERING EDUCATION

- First engineering school (Paris) based on science and math, 1795 – our current model
- British and US – Classical + “shop” courses
- Congress, 1862, established land-grant universities “to promote liberal and practical education”
- Engineering schools grew from 4 to 100 by 1900 using the British model of practical courses

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Assessment Question

[The first metal column design formula was a fit to test results. True or False]

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8. Future

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1900-1944: STRUCTURAL MECHANICS, MATERIALS

No mention of the word *buckling* in the previous centuries

Google – When did the word *buckling* originate in English

Oxford Dictionary:

Buckling - **a smoked herring**

American Century Dictionary **1909**

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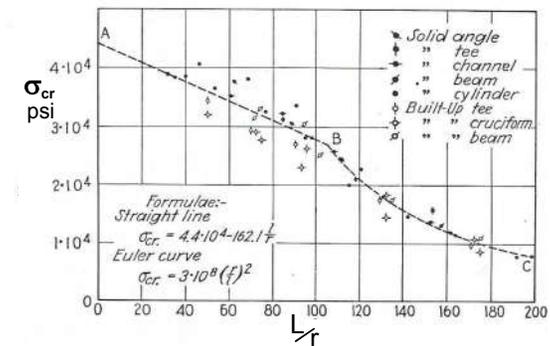
1900-1944: STRUCTURAL MECHANICS, MATERIALS

- BENDING: Slope Deflection, Area-Moment, Moment Distribution
- AXIAL EFFECTS: Stability functions, beam-columns
- Sway frames were a challenge
- Steel (buildings and bridges)-- the W shape arrives

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COLUMN DESIGN: TETMAJER STEEL TESTS (1903)

Straight Line Column Formula



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1900-1944: COLUMN DESIGN

BUILDINGS AND BRIDGES

$L/r < 120$ - elastic Euler formula not applicable

Steel elastic to yield stress

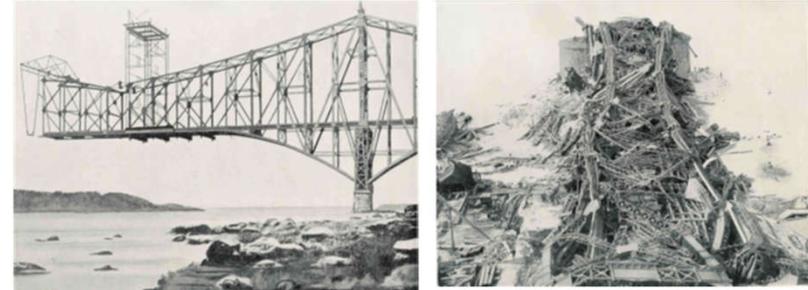
Test scatter due to accidental eccentricities

No perfectly straight members with knife edges

Used eccentricity approach, not buckling

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QUEBEC BRIDGE COLLAPSE (1907)



Major Effect: The beginning of consensus steel design specifications

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ASCE COLUMN COMMITTEES 1909-1933

PURPOSE - Determine the strength and safe working values of steel columns

- Planned and executed a large test program
- Two large test machines -10 and 2.3 million lbs
- 320 tests conducted, mostly flat end, some beam-columns
- No material property tests were conducted

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ASCE COLUMN COMMITTEES 1909-1933

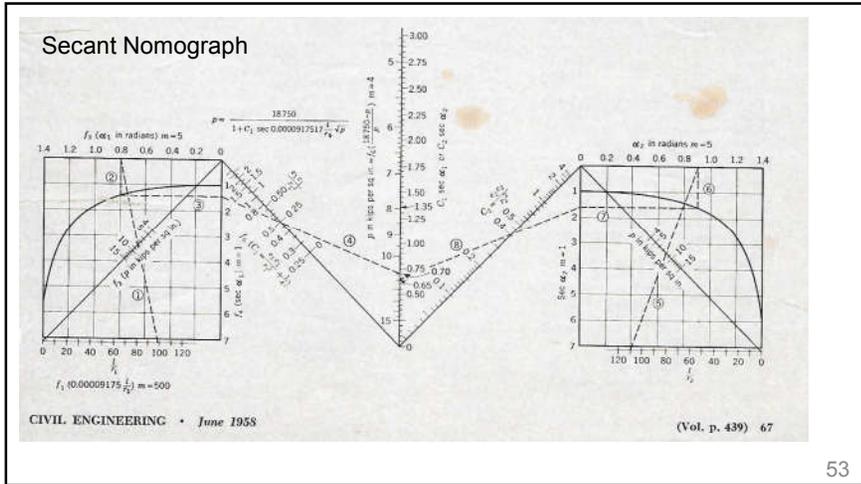
PURPOSE - Determine the strength and safe working values of steel columns

CONCLUSION - Secant formula best adapted to various orientations of end eccentricities compared to Ayrton-Perry

A parabolic formula is a close fit for bridges with a tensile working stress = 18,000 psi for $L/r < 140$

$$F_a = 15000 - 0.25 \left(\frac{L}{r} \right)^2 \quad L/r < 140$$

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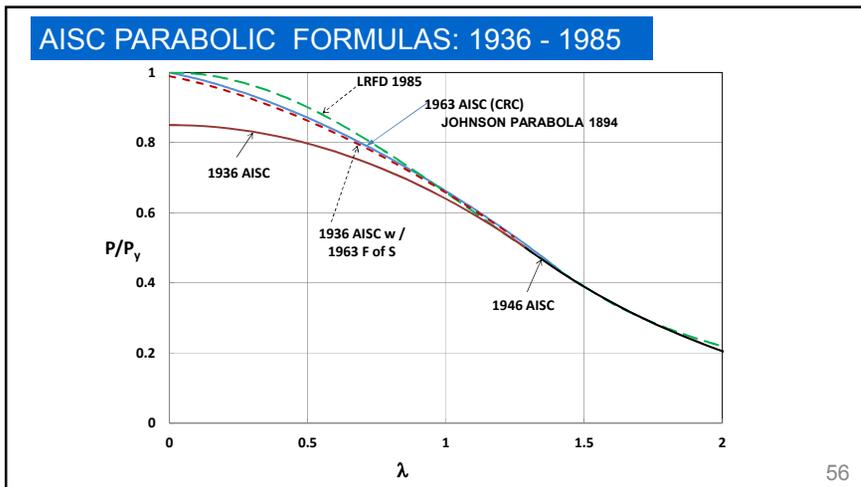
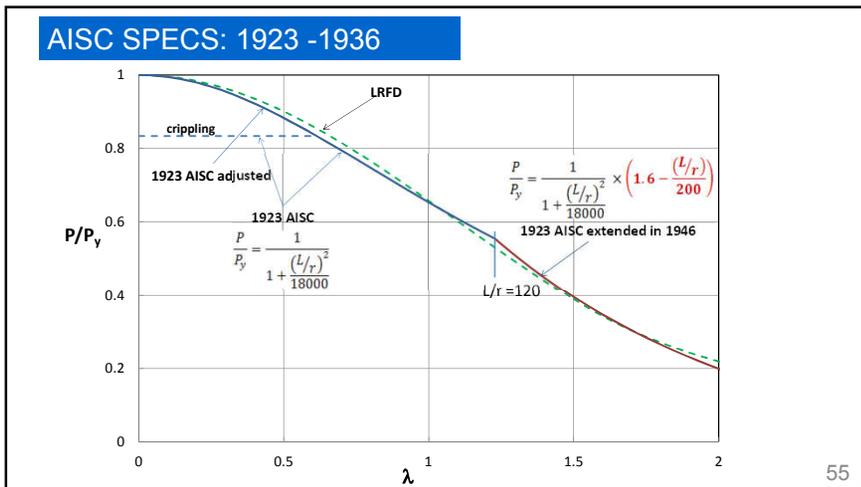


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AISC, est. 1921

- 1st Spec 1923
- 1st Code 1924
- 1st Manual 1925
- 78 pages
- Commentary
- 50 cents

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1936 AISC SPEC

Combined Stresses

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = 1 \quad (\text{Not Derivable})$$

Formerly

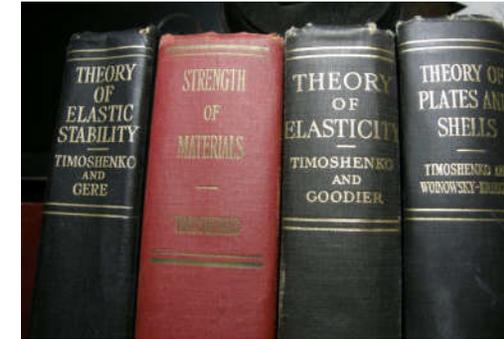
$$f_a + f_b \leq F_a$$

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EDUCATION: S. TIMOSHENKO in US 1922-1972



S. Timoshenko



Universities Morphed to the Math-Science Model

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1944: STRUCTURAL STABILITY RESEARCH COUNCIL

- **SSRC** Originally called Column Research Council (CRC)
- **Goal**
Develop Practical Design Procedures Consistent with Accurate Predictions of Structural Strength
- **First Actions**
Support Bleich's book, *Buckling Strength of Metal Structures*
Focus on the column as part of a real structure

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SSRC

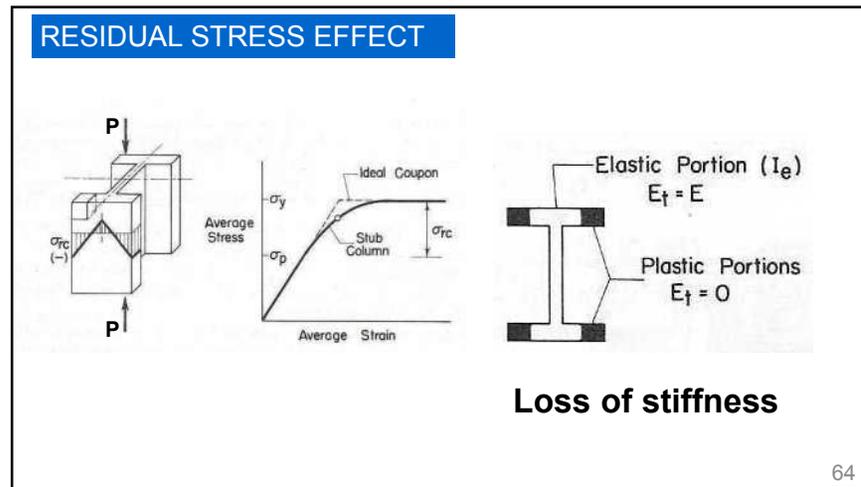
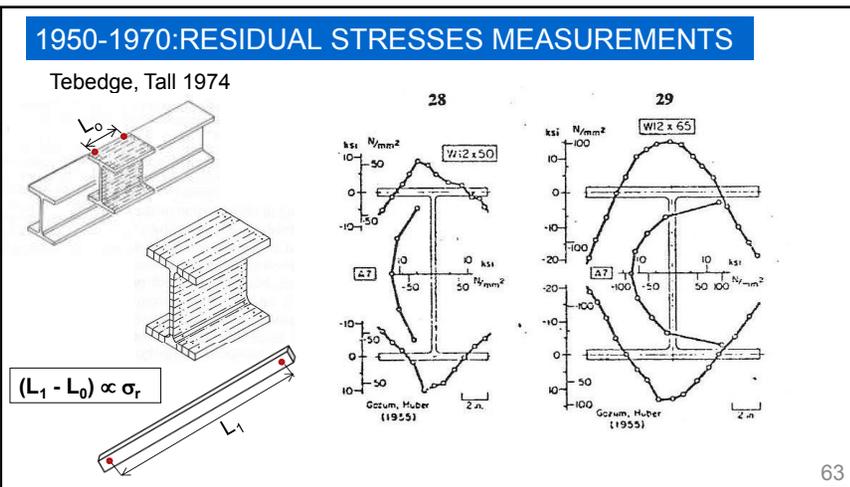
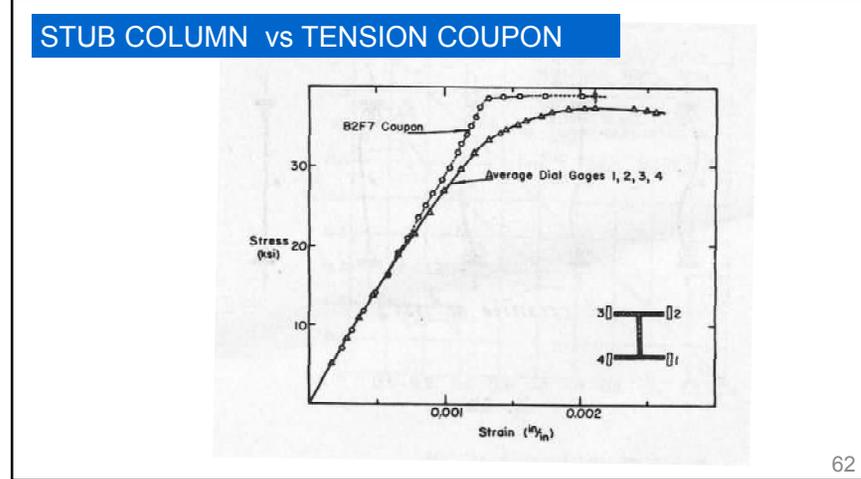
Tangent modulus buckling theory confirmed by Shanley

Whole cross-section compressive tests (stub columns) show a proportional limit due to residual stresses

Residual stress effects can explain column test scatter

Bleich (chapter 1) recommends buckling theory for basis of design and a parabolic design curve

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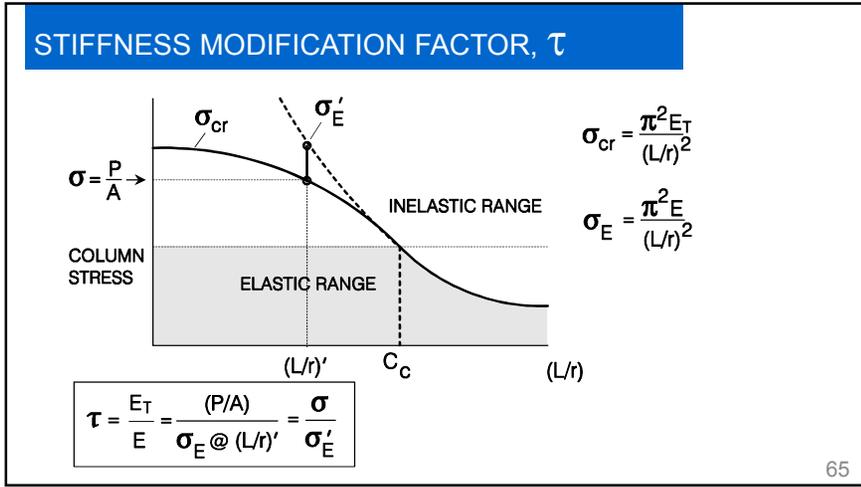
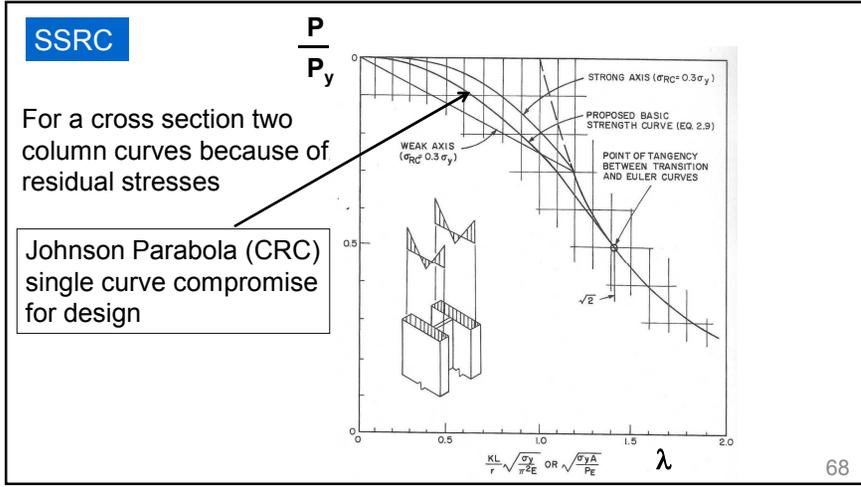
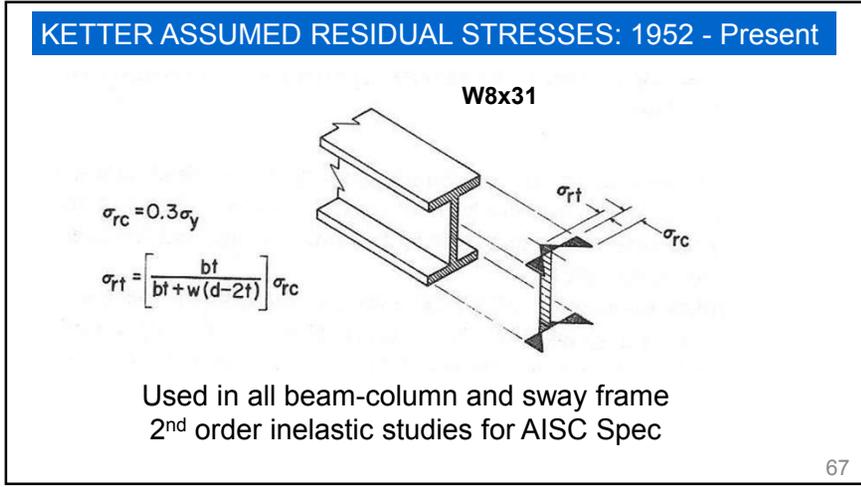
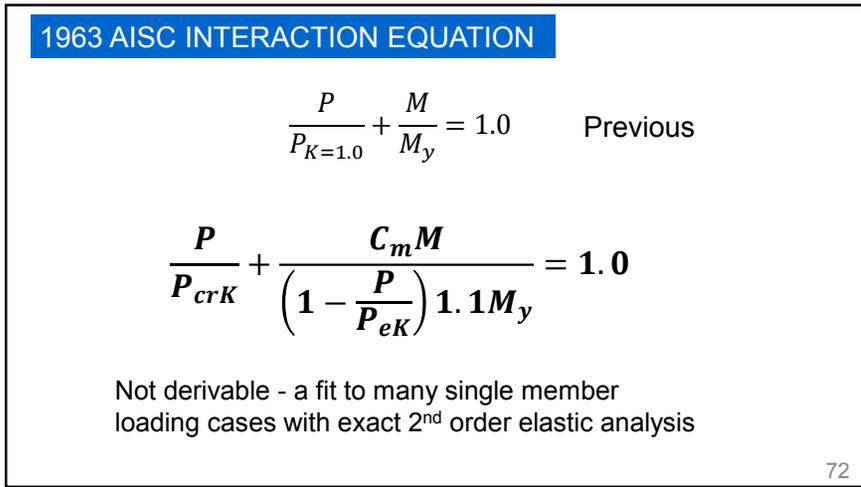
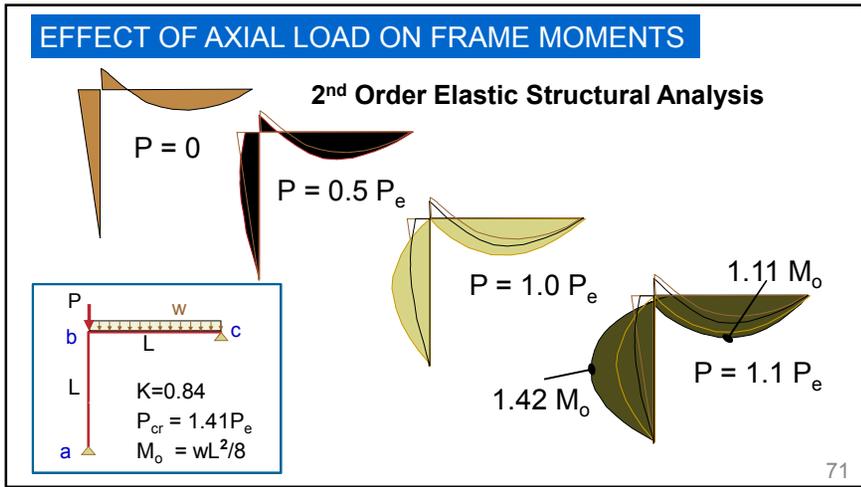
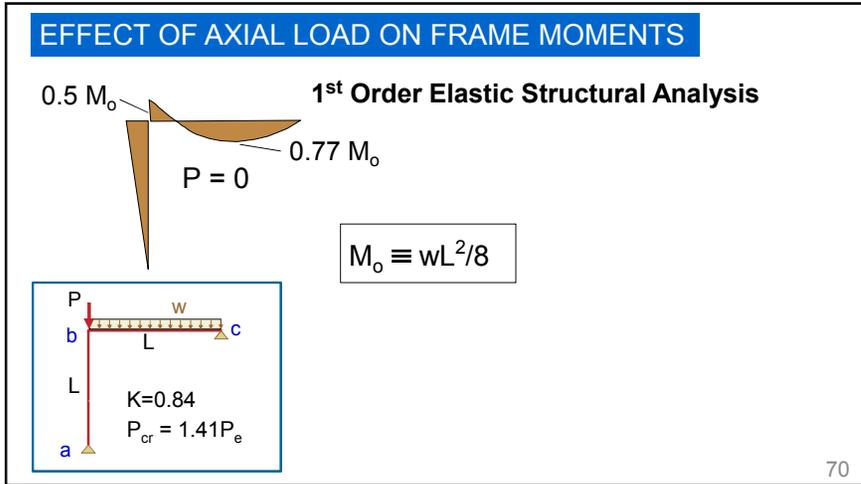
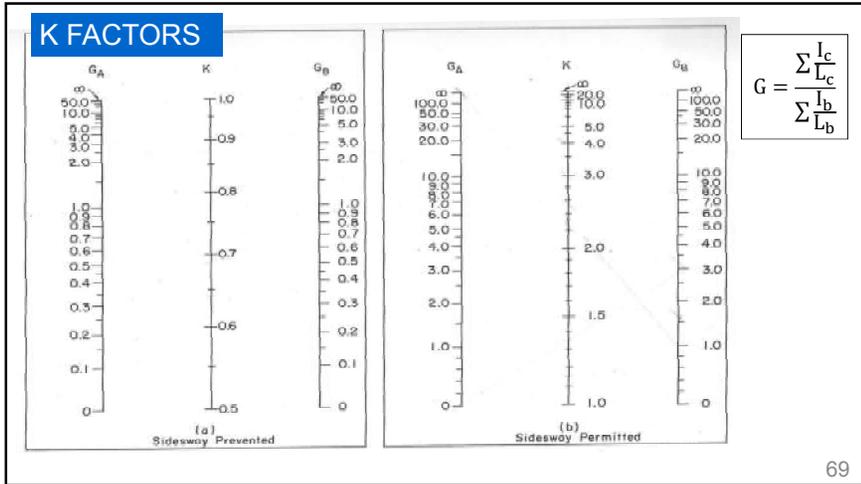
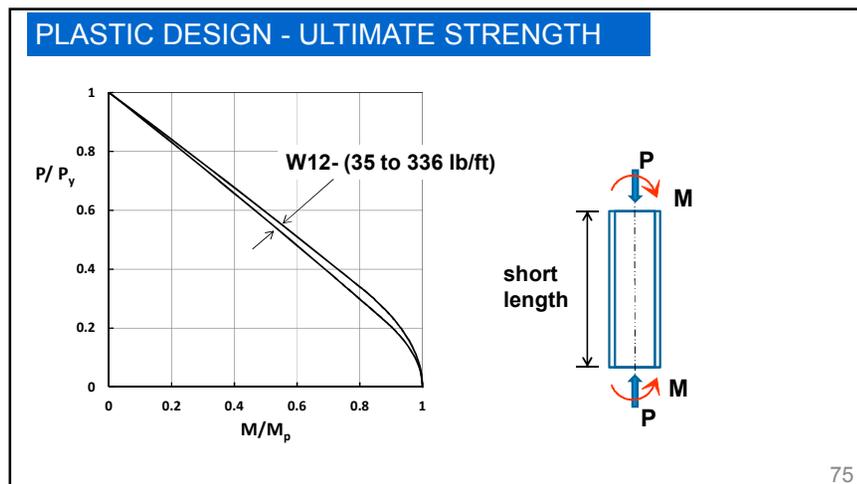
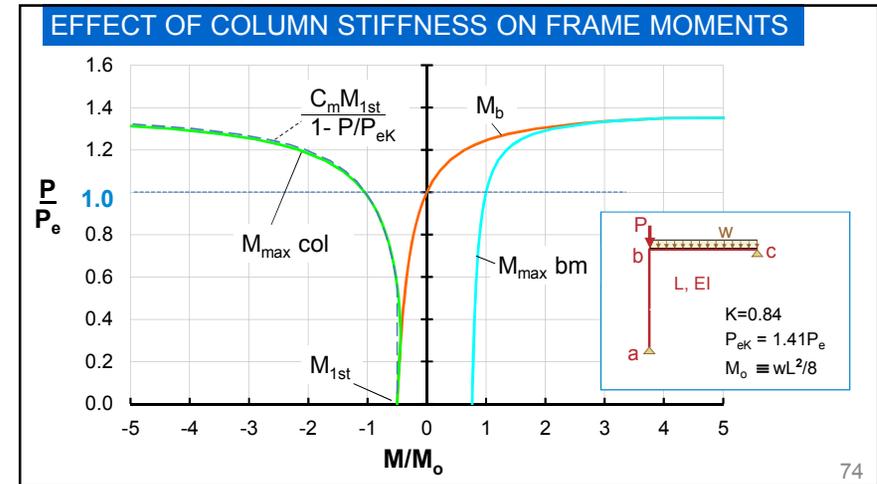
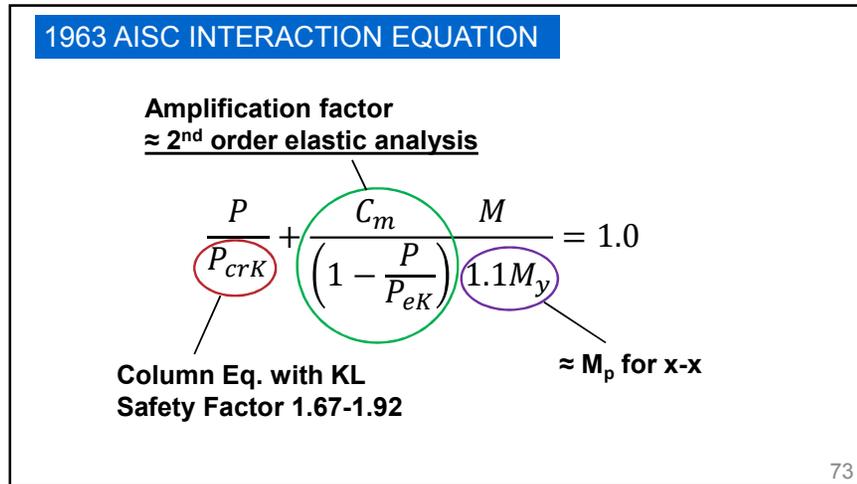


Table 4-21 Stiffness Reduction Factor τ_a 2005 AISC MANUAL

$\frac{P_u}{A_g}$	$\frac{P_e}{A_g}$	F_y , ksi									
		35		36		42		46		50	
ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
45											
44											0.0599
43											0.118
42											0.175
41										0.0262	0.231
40									0.0905		0.285
39										0.153	0.338
38										0.214	0.389
37										0.274	0.438
36						0.0570				0.331	0.486
35						0.194				0.387	0.532
34							0.260			0.441	0.577
33							0.323			0.492	0.620
32					0.0334		0.384			0.542	0.660
31		0.0429			0.115		0.443			0.590	0.699
30		0.127			0.194		0.500			0.636	0.736
29		0.207			0.270		0.554			0.679	0.771
28		0.285			0.344		0.606			0.720	0.804
27		0.360			0.414		0.655	0.0534		0.759	0.835
26		0.431			0.481		0.701	0.148	0.796	0.834	0.863







- ### OUTLINE: (KEY WORDS)
1. ANCIENT – 1650 (Greek temples, calculus)
 2. 1650 -1800 (Euler)
 3. 1800 -1900 (wrought iron, eccentrically-loaded column)
 4. 1900 -1945 (steel frame, AISC)
 5. 1945 -1970 (K-factors, plastic design, residual stress)
 6. 1970 -1985 (frames, multiple column curves, LRFD)
 7. 1985-2018 (K =1.0, 2nd order modified analysis)
 8. Future
- 76

FRAME STABILITY: ΣP CONCEPT

● FOR SWAY BUCKLING OF A STORY

$$\Sigma P_{\text{story column loads}} \leq \underbrace{\Sigma P_{cr_i}}$$

Sway buckling load of each column using alignment chart K

- EACH COLUMN MUST SUPPORT ITS OWN AXIAL LOAD IN THE NO SWAY MODE (i.e. K = 1.0)

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HAND CALCULATOR - 1970



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AVAILABLE TOOLS 1970-1985

Engineering Practice

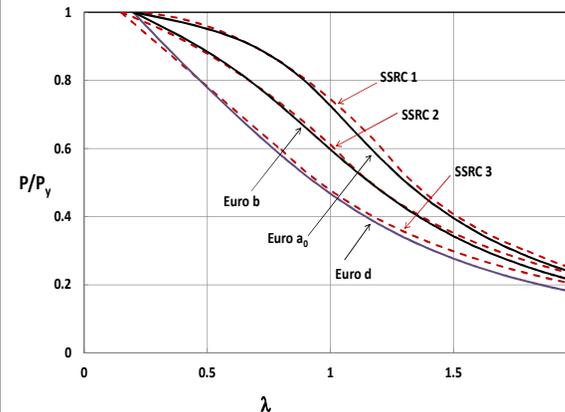
- Hand calculator
- Programs for 1st order structural analysis

University Researchers

- Main frame computers
- Programs for 2nd order inelastic **ultimate strength analysis**
 1. strength of columns including the effects of out-of-straightness and residual stresses
(Buckling approach abandoned for column curve)
 2. braced and unbraced frames with residual stresses
(Beam-column interaction equation)

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MULTIPLE COLUMN CURVES: 1970 - PRESENT



Euro curves are lower bound fits to 1067 pinned end column test results supplemented by numerical simulations.

80

MULTIPLE COLUMN CURVES: 1970 - PRESENT

BS EN 1993-1-1:2005
EN 1993-1-1:2005 (E)

Table 6.2: Selection of buckling curve for a cross-section

Cross section	Limits	Buckling about axis	Buckling curve	
			S 235 S 275 S 355 S 420	S 460
Rolled sections 	$t_f > 1.3$	y-y	a	a ₀
		z-z	b	a ₀
	$40 \text{ mm} < t_f \leq 100$	y-y	b	a
		z-z	c	a
	$t_f \leq 100$	y-y	b	a
		z-z	c	a
$t_f > 100$	y-y	d	c	
	z-z	d	c	
Welded sections 	$t_f \leq 40 \text{ mm}$	y-y	b	b
		z-z	c	c
	$t_f > 40 \text{ mm}$	y-y	c	c
		z-z	d	d
Hollow sections 	hot finished	any	a	a ₀
		cold formed	c	c

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MULTIPLE COLUMN CURVES: 1970 - PRESENT

Adopted by ECCS (Euro Code) in 1978

Concept not considered by AISC Specification

- **Added complexity not practically justified**
- Curves based on effect of residual stresses in columns with pinned ends
- Selection table incomplete and based on limited data
- End restraint greatly diminished the difference between x-x and y-y column response
- **Research on columns in frames (combined axial and bending interaction equations) was based only on a single column curve**

82

1985 LRFD COLUMN

Same as current formula

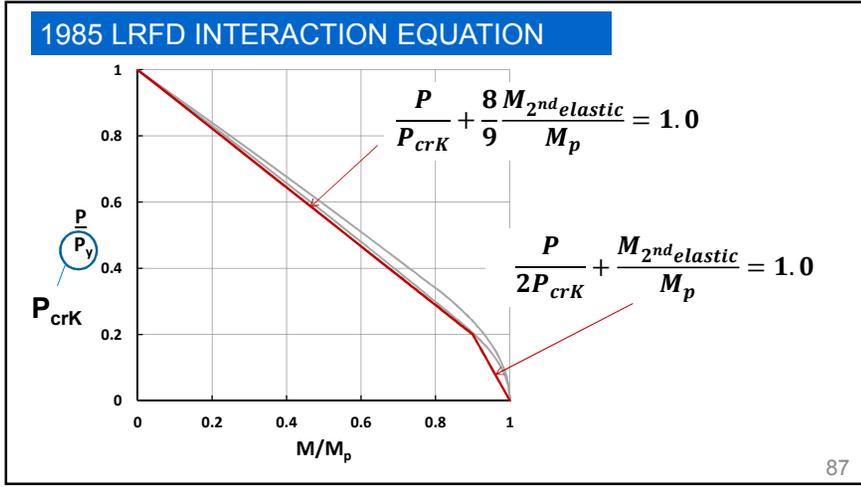
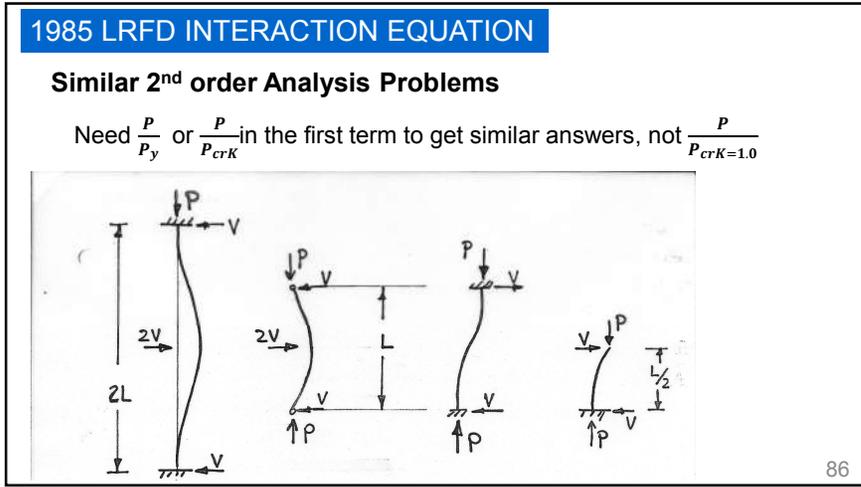
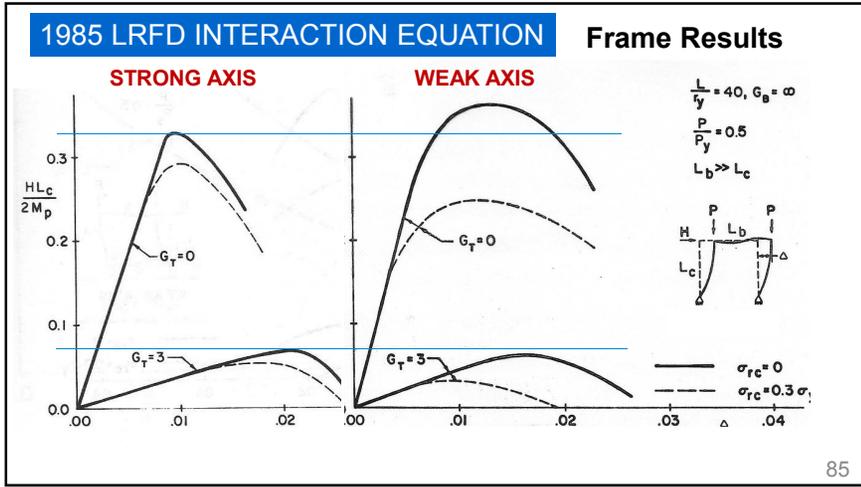
83

1985 LRFD INTERACTION EQUATION

- AISC Stability Task Group spent 8 years in development
- Over 2000 unbraced frames were analyzed for maximum strength
- Columns were initially straight but contained residual stresses
- Three full size two-bay unbraced frames were tested
- Three laterally loaded biaxial restrained beam-columns were tested

84





1985 LRFD INTERACTION EQUATION

$M_{2^{nd} elastic}$ ----- Must include $P\delta$ and $P\Delta$ effects

Can approximate by using $M_{1^{st} order}$ with B_1 and B_2 amplification factors

88

1985 PREDICTION

WHERE ARE WE HEADED

- USE INELASTIC 2nd ORDER ANAL. for $P\Delta_i$
- USE τ I in elastic anal. programs to get Δ_i
- THERE WILL BE AN INTERACTION EQ.

$$1.0 = \frac{P}{P_{yield}} + \frac{M_{sway\ inelastic\ 2^{nd}\ order}}{M_u} + \frac{M_{no\ sway} C_m}{M_u (1 - \frac{P}{\tau P_c})}$$

$\rightarrow B_2$; $M_{1st\ order}$ use τ

strong M_p
weak $< M_p$

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OUTLINE: (KEY WORDS)

1. ANCIENT – 1650 (Greek temples, calculus)
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4. 1900 -1945 (steel frame, AISC)
5. 1945 -1970 (K-factors, plastic design, residual stress)
6. 1970 -1985 (frames, multiple column curves, LRFD)
7. 1985-2018 (K =1.0, 2nd order modified elastic)
8. Future

90

2005 LRFD INTERACTION EQUATION

- Same frames in the 1985 LRFD study were reanalyzed but with an initial column out of straightness = L / 1000.
- Take advantage of available structural analysis programs that perform correct elastic 2nd order analyses.
- Develop a design approach that eliminates effective length

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2005 LRFD INTERACTION EQUATION Frame Result

2nd order inelastic analysis

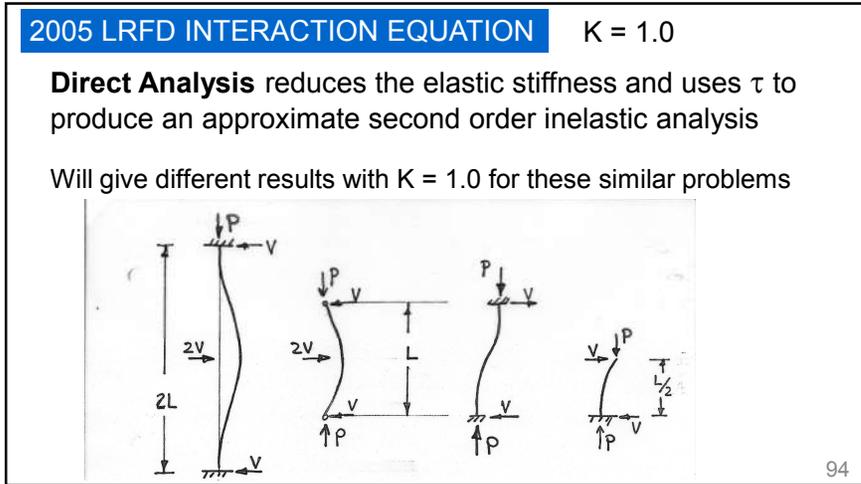
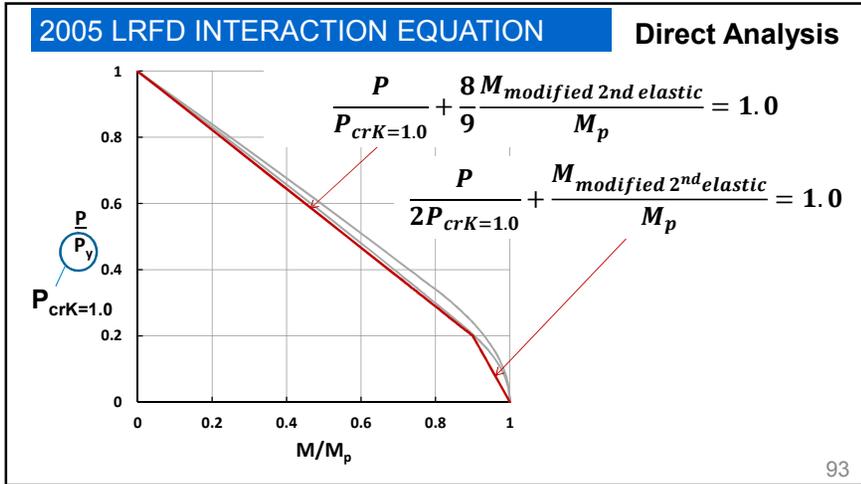
$P/P_y = 0.4$

Maximum Moment

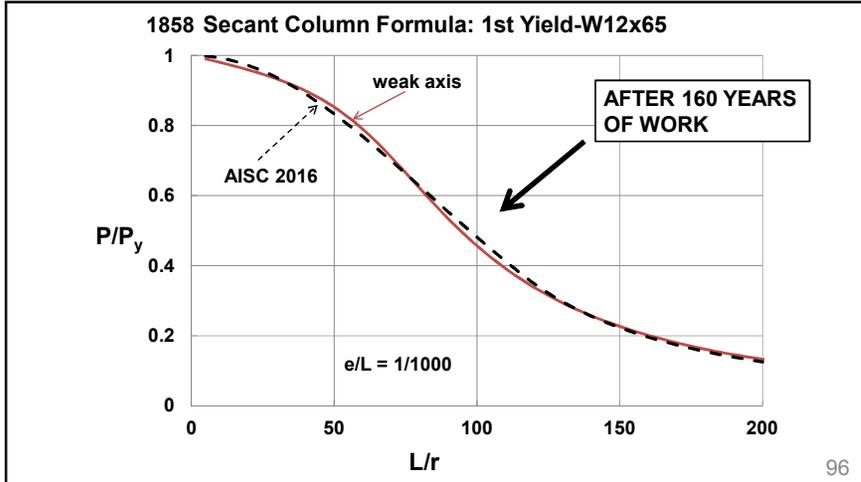
- $\frac{HL}{2} + P\Delta_{inelastic}$ (exact)
- $\frac{HL}{2} + P\Delta_{reduced\ elastic}$ (use K= 1) (direct analysis)
- $\frac{HL}{2} + P\Delta_{elastic}$ (use KL)

no residual stress ———
residual stress = 0.3F_y - - -

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- ### OUTLINE: (KEY WORDS)
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 6. 1970 -1985 (frames, multiple column curves, LRFD)
 7. 1985 -2018 (K = 1.0, 2nd order modified analysis)
 8. **Future**
- 95



CURRENT COLUMN DESIGN

- Based on ultimate strength from a 2nd order inelastic analysis that considers the effects of 1880's vintage column out-of-straightness ($L/1000$) and residual stresses from the 1960's ($0.3F_y$).
- Steel production and manufacturing processes have changed:
 1. Continuous casting in a dog-bone shape
 2. Continuous rolling operation
 3. Cold rotary straightening of most sections

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RESIDUAL STRESSES

Rotary straightening equipment

Before straightening

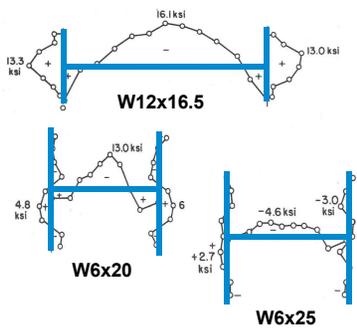


98

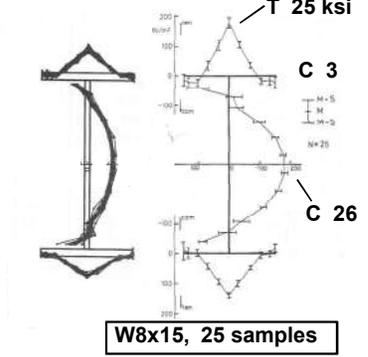
RESIDUAL STRESS

Rotary-Straightened Members

Yura, 1964



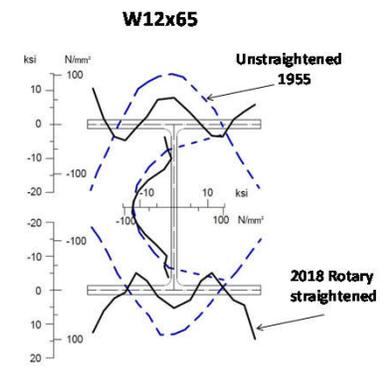
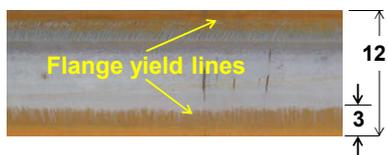
Itoh, 1984



99

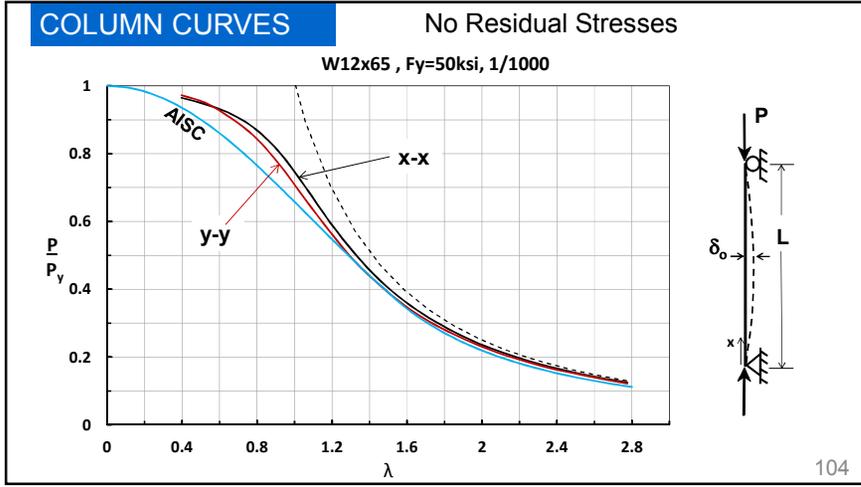
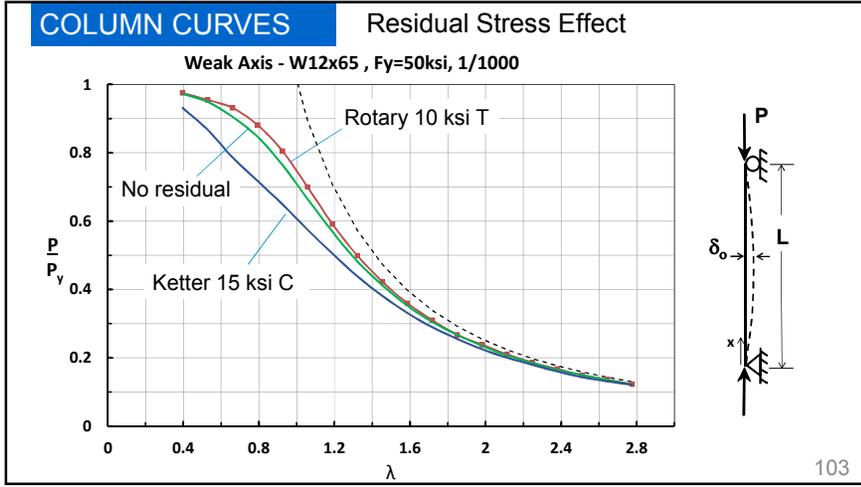
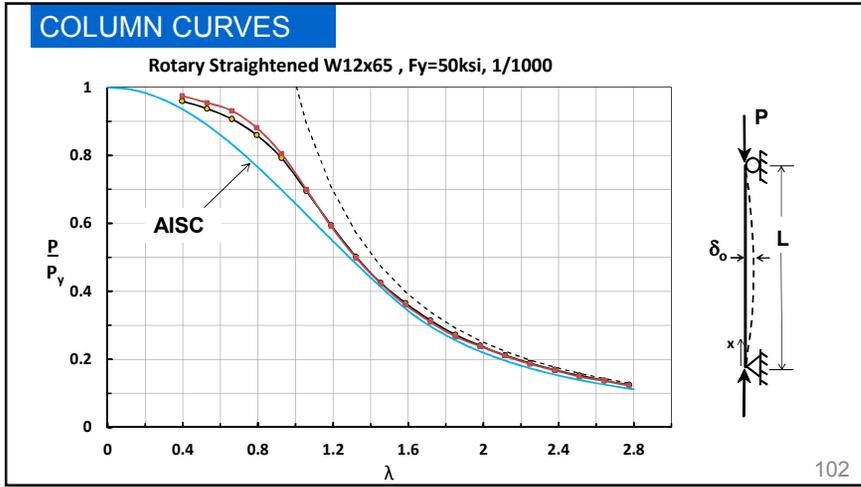
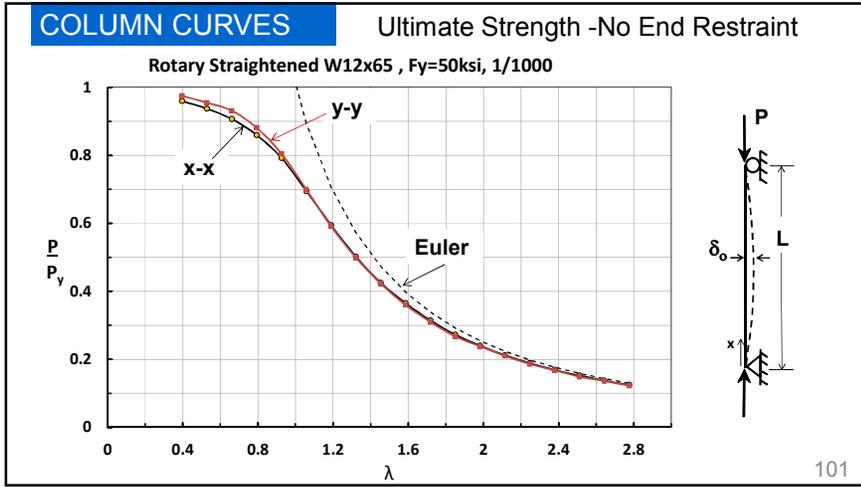
ROTARY STRAIGHTENED 2018

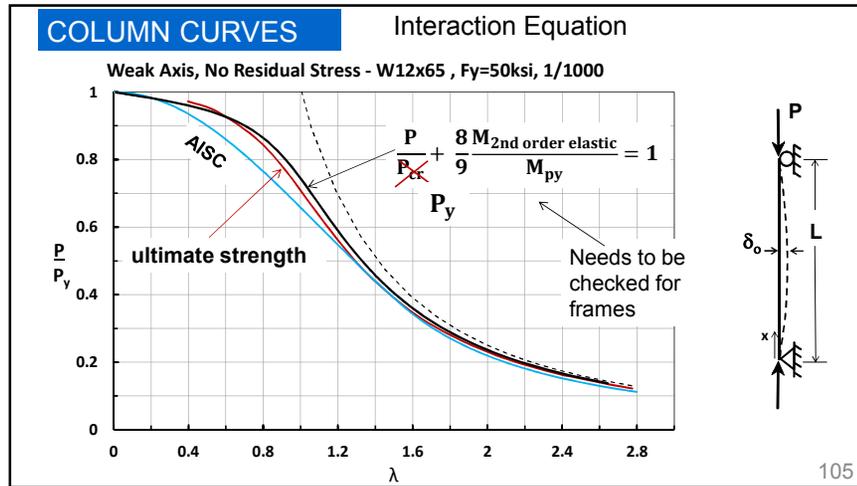
Sections up to W24x370



100







- SUMMARY**
- The column curve itself has changed very little over the past 200 years.
 - The basis of the column curve has transitioned from:
 1. Eccentrically loaded with elastic limit (1820-1961)
 2. Straight member (buckling) with residual stresses (1961-1985)
 3. Ultimate strength with initial out-of-straightness, residual stress and small restraint (1985-present)
 4. Plastic strength with initial eccentricity, no residual stress (maybe)
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SUMMARY Interaction Equations

< 1936	(Axial + Bending) Stresses \leq Column Strength
1936-1961	$\frac{P}{P_{crK=1.0}} + \frac{M_{1st}}{M_y} = 1$ (Not Derivable)
1961-1985	$\frac{P}{P_{crK}} + \frac{C_m M_{1st}}{\left(1 - \frac{P}{P_{eK}}\right) 1.1 M_y} = 1.0$ (Not Derivable)
1985-2005	$\frac{P}{P_{crK}} + \frac{8 M_{2nd}}{9 M_p} = 1.0$ (Not Derivable)
2005-2018	$\frac{P}{P_{crK=1.0}} + \frac{8 M_{modified\ 2nd}}{9 M_p} = 1.0$ (Not Derivable)

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SUMMARY Interaction Equations

Future ? $\frac{P}{P_y} + \frac{8 M_{2nd\ order\ elastic}}{9 M_p} = 1$ (Derivable)

THE END

Thank You for Attending

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AISC | Questions?



Smarter.
Stronger.
Steel.

CEU / PDH Certificates

- You will receive an email on how to report attendance from: registration@aisc.org.
- Be on the lookout: Check your spam filter! Check your junk folder!
- Completely fill out online form. Don't forget to check the boxes next to each attendee's name!



Smarter.
Stronger.
Steel.

CEU / PDH Certificates

- Reporting site (URL will be provided in the forthcoming email).
- Username: Same as AISC website username.
- Password: Same as AISC website password.



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Steel.



AISC | Thank you.



Smarter.
Stronger.
Steel.