DEVELOPMENTS IN CIVIL ENGINEERING, 37

DESIGN OF WELDED TUBULAR CONNECTIONS

Basis and Use of AWS Code Provisions

Peter William MARSHALL

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PREFACE

Although tubular structures are reasonably well understood by designers of offshore platforms, onshore applications often suffer from "learning curve" problems, particularly in the connections, tending to inhibit the wider use of tubes. This book was written primarily to help remedy this situation by the principal author of the AWS D1.1 Code provisions for tubular structures.

The intended audience is users of the Code: designers of offshore platforms, designers of significant onshore tubular structures, and engineers involved in formulating company guidelines for these applications. Writers of other codes and graduate students and researchers in the area of tubular structures will also find it useful as a source of background material.

This book is intended to be used in conjunction with the AWS Structural Welding Code - Steel, AWS D1.1-90, published by the American Welding Society, Miami. It relies on the use of Code material which is not reproduced herein.

The manuscript was prepared as a PhD dissertation for the Department of Architecture, Kumamoto University, Kumamoto, Japan. The author is grateful to his committee chairman, Professor Yoshiaki Kurobane for inspiring this effort, and to Professor Joseph A. Yura, University of Texas, and Professor Jaap Wardenier, Delft University of Technology, for their input and guidance during the preparation of the manuscript. Charles Spitzfaden and Yolanda Estrello assisted with drafting and word processing, respectively, and Joop Paul proofread the completed work.

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Chapter 1

INTRODUCTION TO TUBULAR STRUCTURES

1.1 ATTRIBUTES OF TUBES

Tubular members benefit from an efficient distribution of their material, particularly in regard to beam bending or column buckling about multiple axes. For architecturally exposed applications, the clean lines of a closed section are aesthetically pleasing, and minimize the amount of surface area for dirt, corrosion, or other fouling. Simple welded tubular joints can extend these clean lines to include the structural connections. With circular tubes, reduced drag forces also apply for wind, waves, and blast loadings.

1.2 ARCHITECTURAL AND STRUCTURAL FORMS

1.2.1 Onshore Applications

Tubular columns are extensively used in high-clearance single story buildings, such as shopping malls and warehouses. Here radius of gyration is more important than section area, and the connections are simple and straightforward--fillet welded base plates and shear plates for bolting to beam webs.

Tubular designs are also widely used for lightweight long span structures, such as expressway overhead signs, pedestrian bridges, booms for construction cranes and mining draglines, drilling derricks, radio masts, and the like. They have also been proposed for orbiting space stations.

Tubular space frames are increasingly finding use in such dramatic and monumental architectural applications as long-span roofs, atrium skylights, radio-telescope dish antennas, Olympic ski-jumps, space-shot launching towers, and spectacular looping amusement park rides. Like other rolled shapes, rectangular tubes offer simple welded connections in orthogonal planes. However, for the truly unusual structure, circular tubes offer simple welded connections in any orientation desired.

Unfortunately, the potential elegance of these structures is often spoiled because of problems with the connections. The designer may lack confidence in simple direct welded connections, and devise an awkward, ugly gusseted joint to do the same job. The fabricator may be unprepared for the specialized layout, cutting, fitting, welding, and inspection tasks involved. The erector may require bolted field connections. Finally, the project may become embroiled in a dispute with officials who are also not fully prepared to deal with the specialized technology involved.

Solutions to these problems are covered by the "Tubular Structures" section of the American Welding Society D1.1 Structural Welding Code - Steel. Much of the technology from which this part of code evolved was developed by the offshore oil industry, as reflected in the parallel provisions of API RP 2A, Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms.

1.2.2 Offshore Applications

Thousands of large tubular structures have been built for offshore oil drilling and production in the last forty years. The typical structure consists of a tubular space frame, or jacket, which extends from the seafloor to just above the sea surface. This is usually fabricated in one piece onshore, transported by barge, launched at sea, and upended on site by partial flooding. Tubular piling are driven through the jacket legs to resist vertical gravity loads and

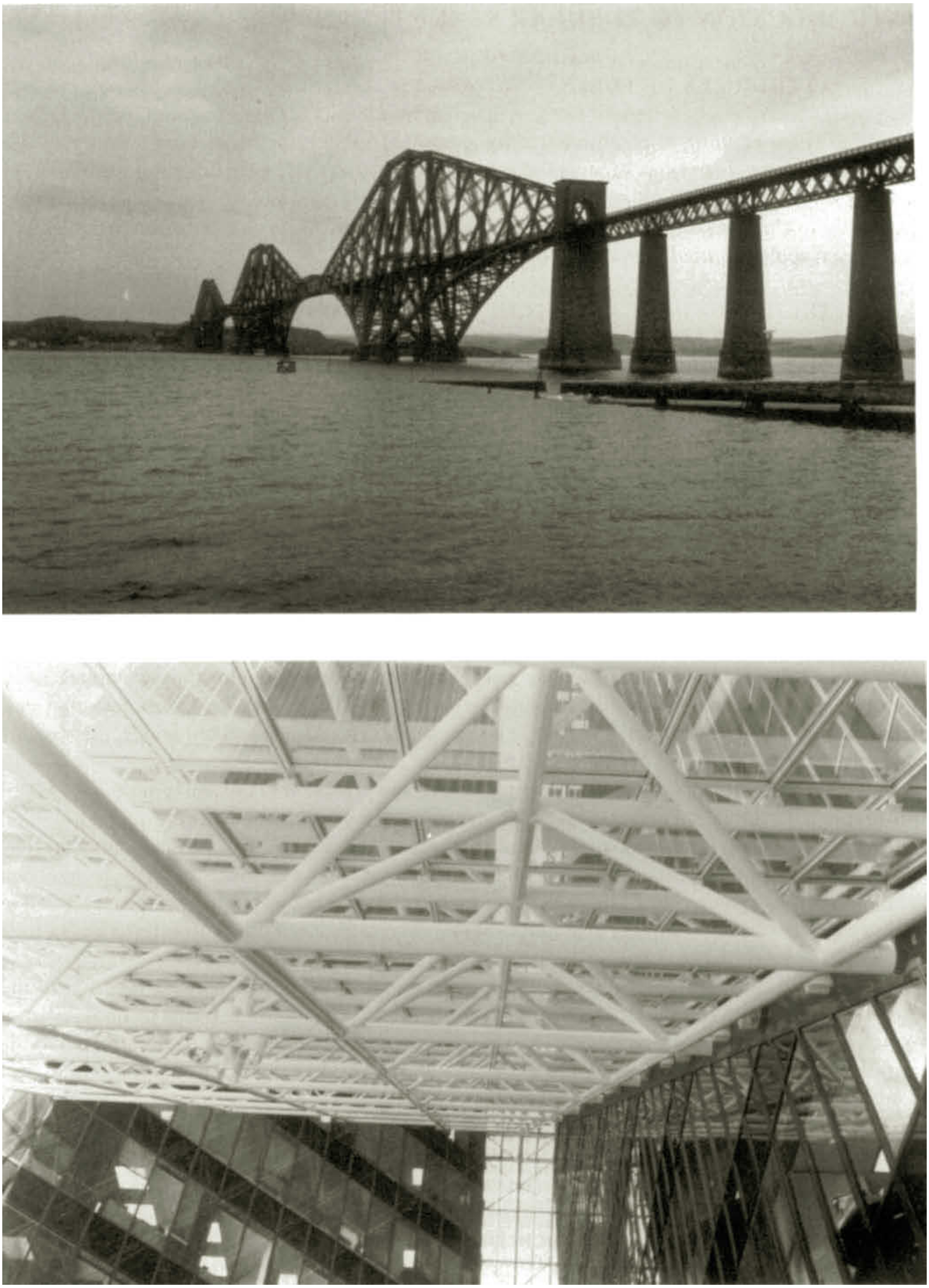

Fig. 1.1. Onshore applications of tubular structure. (a) Firth of Forth railway bridge, Scotland, 1880's. (b) Atrium space frame, Houston, 1980's.

lateral storm loads. To complete the structure, a working deck section is added, usually a composite of tubular members and conventional rolled sections (ref. 1).

Tubular construction is also used for the lattice legs of jack-up mobile drilling units, and for the interconnecting space frame of column- stabilized semisubmersibles, a class of floating drilling rigs.

Early development of offshore technology was largely a trial and error experience. Structural design was not so much governed by official regulations as it was by the desire on the part of offshore operators to protect their own considerable investment. The collapse of even a small drilling/production platform involved a loss of tens of millions of dollars--including, in addition to the structure itself, equipment, wells, clean-up costs, and loss of income. For today's deepwater structures, the loss can exceed $1 billion. Because a degree of uncertainty exists in both the strength of structures and the magnitude of applied loads, the risk of structural failure is not totally eliminated by the inclusion of a safety factor. Rather, an attempt is made to select design criteria on a rational economic basis; that is, to minimize the sum of first cost plus deferred future risks (ref. 2).

In making such trade-offs, the optimum point is not sharply defined; thus calculation of the probability of failure need not be absolutely precise in order to serve its purpose. Furthermore, the reliability viewpoint provides a useful rationale, in that it forces one to examine the bias and uncertainty at each step of the way. This rationale has proven useful in interpreting research results and defining the design criteria we now use. Finally, there are social constraints present which make it unpalatable to make trade-offs between dollars and human safety or environmental pollution. The safety index is a useful measure of structural reliability for this purpose, without the legal, social, and psychological impact of probabilities of failure.

We can define the safety index as the expected value of the margin between real load and real resistance, expressed in units of the standard deviation of total uncertainty. For onshore public structures, the safety index ranges from 2.5 to 4.0, and failures are so rare that their statistics are not well defined. For new offshore platforms designed for the 100-year storm, the safety index ranges from 2 to 3 in terms of the lifetime risk of overload failure; the corresponding average annual loss rate is on the order of 0.1% or less. This is low enough that overload is not the dominant risk; blowouts, fires, and collisions account for more of the catastrophic losses.

Offshore structures were not always this reliable. Early joint design consisted of the instruction: "cope to fit and weld solid". Tubular braces were simply welded to the jacket legs, which served as the main member at the tubular connection without any reinforcement. After several hurricanes, recurring failure modes became apparent in these simple connections. As will be discussed in subsequent chapters, these include local punching-shear/pullout failure in the main member, general collapse of the main member, progressive failure of the weld, and lamellar tearing. Materials problems were also experienced, including poor weldability and brittle fracture. Although fatigue failure has been an ongoing concern of research over the last 20 years, this geriatric mode of failure has only recently begun to be observed in actual structures.

1.3 THE NEED FOR AN INTEGRATED APPROACH

Despite the availability of codes of practice like AWS D1.1, welded structural connections in tubular space frames have developed a certain mystique. This is no doubt enhanced by a number of spectacular problems which have occurred. A few have resulted in structural collapse, while many others spelled financial disaster for the contractor involved. Often, when a welded tubular connection fails, the fracture is in the heat affected zone at the toe of the weld joining a branch member or attachment to the main tube. The designer involved may seize upon this fact to attribute the failure to faulty materials or welding, and elaborate metallurgical witch hunts may be staged to bolster this claim. Never mind that the weld toe is

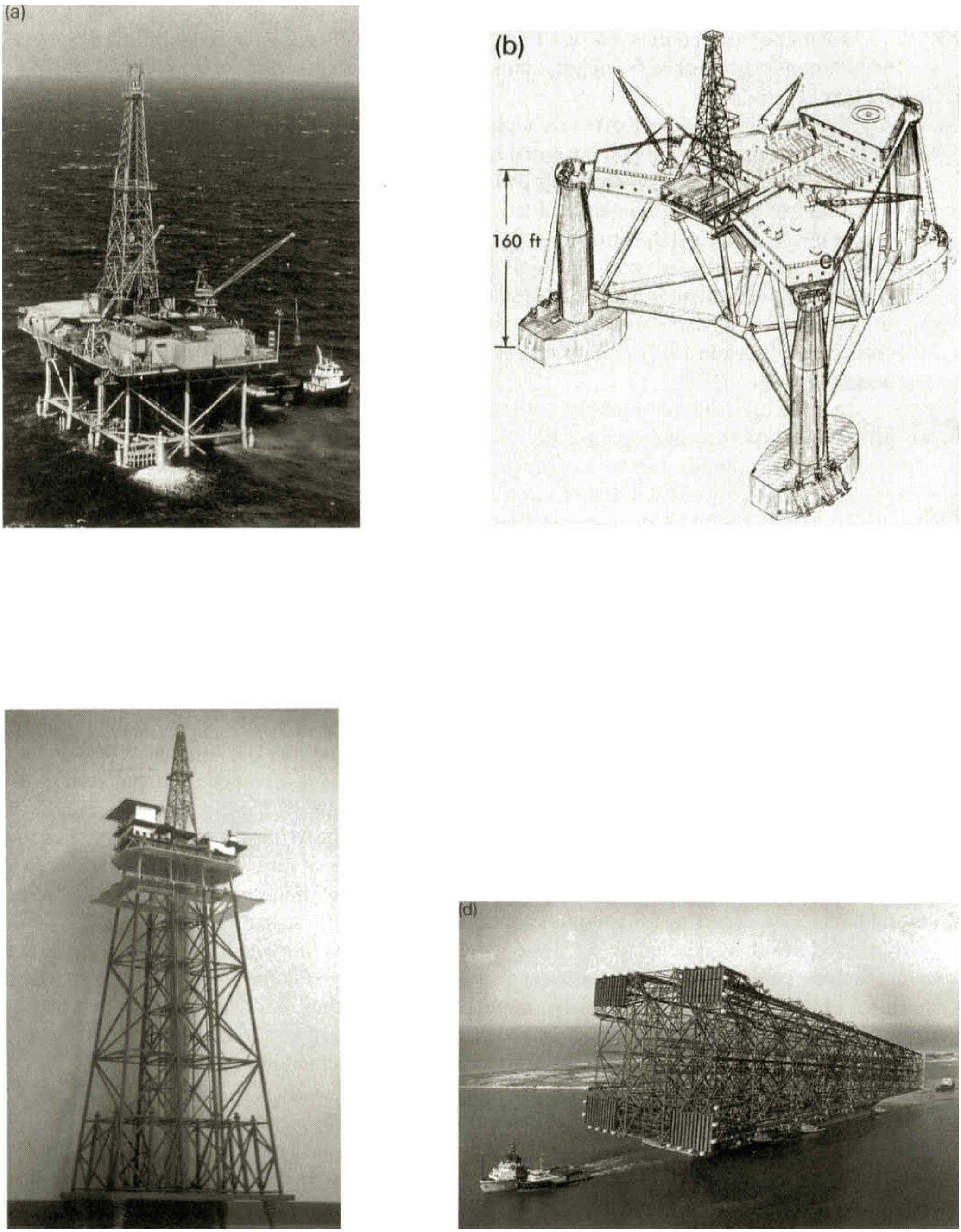

Fig. 1.2. Offshore applications of tubular structure. (a) Topsides of self-contained drilling and production platform. (b) Space frame of semi-submersible drilling rig. (c) Fish-eye view of 8-leg platform for 100m water. (d) Bullwinkle jacket for 400m water.

also the site of stress concentrations which are so high that most practical connections experience localized plastic straining before reaching their design load. The lawyers and their expert witnesses get rich, and the mystique grows.

Perhaps to a larger degree than with other structural forms, welded tubular connections require an integrated approach to fracture control. Design, material selection, fabrication, welding, and inspection must all be considered--and they are interrelated. Responsible design includes more than using stress analysis calculations to dimension the main structural elements. Connections require equal attention, if not more. The designer must understand the demands he implicitly places on the materials to be used, e.g., ductility as well as yield strength and availability; and he must anticipate the methods of fabrication and welding, their limitations, and their effects on service performance. The designer who blindly uses the code formulas is a failure waiting to happen. If only to protect themselves, the practical materials and welding people who follow in executing his design should also understand what demands are being placed upon their part of the overall fracture control picture.

1.4 AUTHOR'S VIEWPOINT FOR THIS MONOGRAPH

The architecture of tubular structures has fascinated the author through his career as a structural engineer. "Architecture" is defined as the art and science of designing and successfully executing structures in accordance with aesthetic considerations and the laws of physics, as well as practical and material considerations.

Onshore, where tubular structures are often exposed for dramatic effect, it has often been painful to see grand concepts fail in execution due to problems in the tubular joints, or structural connections. Such "failures" range from awkward detailing, to "learning curve" problems during construction, to excessive deflections or collapse.

Offshore, the oil industry went through the painful stage about 20 years ago. Research, testing, and practical applications have progressed to the point where tubular connections are about as reliable as the other structural elements which engineers normally deal with. The author participated in the resolution of the problem areas, synthesizing and putting into practice the research of such pioneers as Toprac, Bouwkamp, and Pickett. His joint designs and design procedures are part of most of Shell's large Gulf of Mexico platforms, including the world record Bullwinkle jacket in 1350-ft. water depth, as well as the Brent "A" platform offshore from Scotland (famous for its widely quoted "North Sea Brent" crude oil price marker).

The art and science of welded tubular connections which emerged from this effort has been codified in AWS D1.1 (ref. 3). This Monograph will describe, from the viewpoint of a primary participant, the conceptual basis and historical development of the code, including recent revisions. It draws heavily on the author's previous work, notably the 1984 Houdremont lecture (ref. 4), and on his three chapters in McClelland's book on offshore platforms (ref. 5).

Although there will be updating and expansion upon the previous work, and an effort to compare the Code with some of the voluminous new data coming forth, no claim of comprehensiveness in this regard is made. Recent, more exhaustive reviews of the worldwide data base can be found in Wardenier (ref. 6) and in Billington, Tebbett, and Lalani (ref. 7).

Similarly, this work will focus on tubular connections, rather than design of tubular members, save for the broad remarks which follow. Fully detailed background and justification for these would take up another book.

1.5 TUBULARS AS STRUCTURAL MEMBERS

API Recommended Practices for the Planning, Designing, and Constructing of Fixed Offshore Platforms, API RP 2A, (ref. 8), gives detailed guidance for tubular structures as used offshore. With few exceptions, structural steel design follows the basic allowable stresses of the AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings, extending these criteria to tubular members.

The AISC Steel Construction Manual (ref. 9) lists dimensions and section design properties for a number of tubular sections. Standard weight, extra strong, and double extra strong circular sections from half-inch to 12-inch nominal diameter are widely available from stock, particularly in mild steel grades, 35 to 36 ksi yield strength (246 to 253 MPa). In the U.S., commonly used larger sizes include diameters and wall thicknesses as listed in Table 1.1.

In offshore practice, still larger sizes are custom fabricated from plate, typically in 6-inch (152mm) increments of diameter and 0.125-inch (3mm) increments of wall thickness. Diameter/thickness ratios commonly range from 20 (a limit for cold-straining) to 60 (a limit for local buckling).

TABLE 1.1 PROPERTIES OF COMMONLY USED SIZES OF STRUCTURAL PIPE

O.D. INCHES	WALL THICK. IN.	AREA SQ. IN.	WEIGHT LB/FT	MOM. OF INERTIA IN.-4TH	SECTION MODULUS IN.-3RD	RADIUS GYRATION IN.
6 5/8	.280	5.58	19.0	28.1	8.4	2.24
6 5/8	.432	8.40	28.6	40.4	12.2	2.19
6 5/8	.562	10.70	36.4	49.6	14.9	2.15
8 5/8	.322	8.39	28.6	72.4	16.8	2.93
8 5/8	.406	10.48	35.6	88.7	20.5	2.90
8 5/8	.500	12.76	43.4	105.7	24.5	2.87
8 5/8	.718	17.83	60.6	140.5	32.5	2.80
10 3/4	.365	11.90	40.5	160.7	29.9	3.67
10 3/4	.500	16.10	54.7	211.9	39.4	3.62
10 3/4	.593	18.92	64.3	244.8	45.5	3.59
12 3/4	.375*	14.57	49.6	279.3	43.8	4.37
12 3/4	.500	19.24	65.4	361.5	56.7	4.33
12 3/4	.687	26.03	88.5	475.1	74.5	4.27
14	.375*	16.05	54.6	372.7	53.2	4.81
14	.438*	18.66	63.4	429.4	61.3	4.79
14	.500	21.20	72.1	483.7	69.1	4.77
14	.750	31.21	106.0	687.3	98.1	4.69
16	.375*	18.40	62.6	562.0	70.2	5.52
16	.438*	21.41	72.8	648.7	81.0	5.50
16	.500*	24.34	82.8	731.9	91.4	5.48
16	.656	31.62	108.0	932.3	116.5	5.42
18	.375*	20.76	70.6	806.6	89.6	6.23
18	.500*	27.48	93.4	1053.1	117.0	6.18
18	.625	34.11	116.0	1289.0	143.2	6.14
20	.375*	23.12	78.6	1113.4	111.3	6.93
20	.500*	30.63	104.0	1456.8	145.6	6.89
20	.593*	36.15	123.0	1703.7	170.3	6.86
20	.812	48.94	166.0	2256.7	225.6	6.79
24	.375*	27.83	94.6	1942.3	161.8	8.35
24	.500*	36.91	125.0	2549.3	212.4	8.31
24	.687*	50.31	171.0	3421.2	285.1	8.24
24	.750*	54.78	186.0	3705.4	308.7	8.22
24	.968	70.04	238.0	4652.6	387.7	8.15
24	1.000	72.25	246.0	4787.0	398.9	8.13

NOTE: 1 INCH = 25.4mm

*D/t of 30 to 60; semi-compact section (limited plastic rotation capacity)

The AISC manual also lists a large number of square and rectangular sections and their design properties. However, some of the sections listed have only limited availability. Again, larger sections can be fabricated from plate.

1.5.1 Columns

Realistic design for axial compression must reflect the fact that the strength of actual columns is significantly below both of the two theoretical bounds -- yield and elastic buckling. This departure is due to variations in material properties (static yield strength versus the conventional rapid tension test) and imperfections (centerline crookedness, out-of-roundness, and misalignment of adjacent material at butt joints), as well as residual stress.

The AISC design curve, and the original CRC column curve upon which it is based, reflect such considerations and are based on a large number of column tests, representing a variety of sections--hot rolled and welded shapes; open, closed, and solid sections; and both mild and high strength steel; as shown in Figure 1.3(a).

Large tubular columns were not well represented in the original data base. Welded tubes differ from hot rolled sections in possessing significant residual stresses, which promote earlier yielding and lower column strengths. Figure 1.3(b) shows the pattern of residual stresses in a welded box column and a fabricated tube (ref. 10). In addition to the mean longitudinal stresses shown, circumferential residual stresses due to cold forming of the plate also exist, varying through the thickness in a pattern typical of plastic bending followed by springback, for the circular tube.

Column behavior for the fabricated box sections falls significantly on the unsafe side of the CRC curve as shown in Figure 1.4. Tests on small cold formed circular tubes also suggested a lower design curve (ref. 11). Faced with this, the author prevailed upon API to sponsor a series

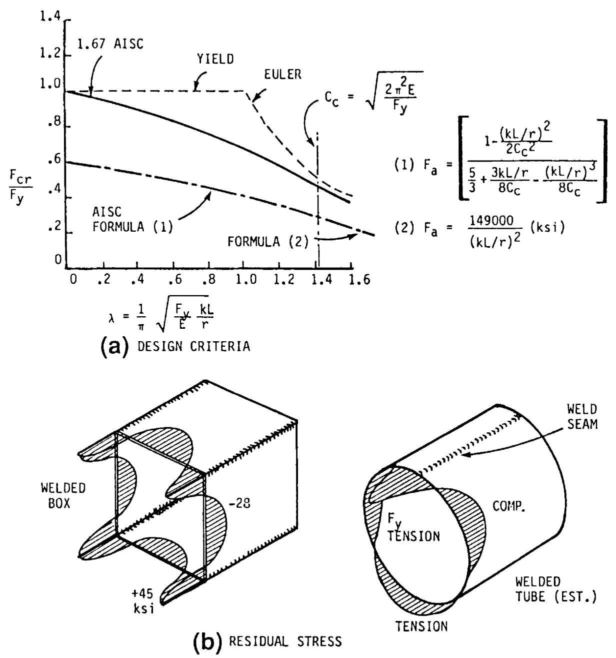

Fig. 1.3. Column stability considerations for tubular structures (from ref. 10).

of tests on fabricated pipe columns at Lehigh University, results of which are also shown (ref. 12). The large range covered by each data plot indicates the range of ambiguity in test interpretation, due to differences between static and conventional dynamic yield strengths, and to friction in the spherical end bearings affecting the effective column length.

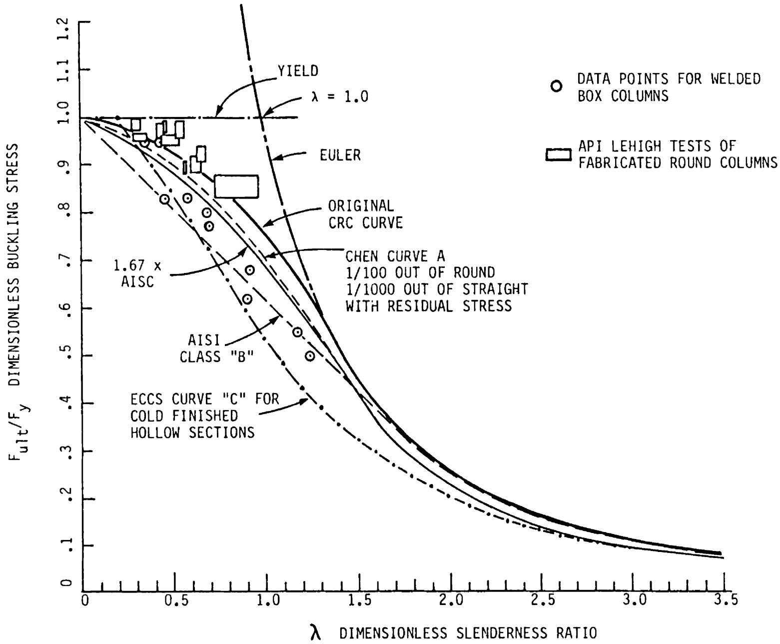

Fig. 1.4. Column buckling curves.

Using advanced analytical methods, Chen et al were able to match experimental test results within a few percent (ref. 13), when actual imperfections and residual stresses in the test specimens were taken into account. Chen then used this same analytical method to produce curve "A" in Figure 1.4, for members just meeting code fabrication tolerances. Since this falls remarkably close to the 1.67 times the AISC design criteria, offshore design practice continues to follow AISC.

The author has not had a similar degree of involvement with criteria development for square and rectangular hollow sections. Most such sections currently available in the U.S. are cold finished. This raises the tensile yield strength, but produces a "round house" stress-strain curve and complex residual stress patterns, so that the relative column behavior is less favorable. American (ref. 14) and European (ref. 15) sources suggest the use of lower column design curves for this application, as indicated by the AISI and ECCS curves in Figure 1.4. A Canadian review of over 300 tests (ref. 16) also suggests the use of multiple column curves, depending on the method of tube manufacture.

Tubular struts with welded end connections enjoy a degree of end fixity which permits the use of effective length factors "k" less than unity. For example, API RP 2A recommends "k" of 0.8 for primary bracing which frames into the larger, stiffer legs of offshore jackets, using connections which substantially match the strength of the sections joined. For other types of tubular structures, applicability of "k" factors less than unity will largely offset the penalty of having a lower column design curve. See Table 1.2.

Although the AISC code permits columns with slenderness ratios, kL/r, up to 200, circular tubular members subject to wind action should observe lower limits in order to avoid vortex induced vibrations. The traditional limit for offshore jackets is kL/r of 120; this corresponds to a critical wind speed of 18 mph (8m/s) and suffices for short construction periods at sites that are not too windy. Members violating this limit frequently vibrate, and some have suffered fatigue cracks. Theoretically, dense members with a lot of damping should be able to withstand wind speeds above critical, without excessive vibration. However, welded members have very low damping, as low as 0.1% of critical, so that only members having D/t ratios less than 16 would be dense enough to avoid the problem. For windy construction sites, with consistent winds of 30 mph (14m/s), a few members with kL/r greater than 90, and D/t of 30 to 60, have encountered vibration problems. Slenderness ratios, kL/r, of 60 or less would be required for lifetime exposure to winds having sustained speeds of up to 70 mph (60m/s), especially for members having low density (high D/t).

TABLE 1.2 EFFECTIVE LENGTH FACTOR k

SITUATION	AMERICAN (REF. 8)	OVERSEAS (REF. 15)
CHORD OF TRUSS IN-PLANE	1.0 TO NODES	MAY BE < 1.0 CONSIDERING RESTRAINT
CHORD OF TRUSS OUT-OF-PLANE	1.0 TO BRACING POINTS	PROVIDED BY WEB MEMBERS (REF. 28)
WEB MEMBERS IN-PLANE	0.8	0.7
WEB MEMBERS OUT-OF-PLANE		
TUBULAR CHORDS	0.8	0.7 W/OVERLAP, β > 0.6 (REF. 29)
OPEN SECTION CHORDS	1.0	
X-BRACES	0.9 OF SHORTER HALF, COUNTER IN TENSION	
SECONDARY BRACING	0.7	
PORTAL SIDESWAY COLUMNS	> 1.0 USE AISC ALIGNMENT CHART	

1.5.2 Bending

(i) **Circular**. In the range where structural pipe may be treated as a compact section--that is, no local buckling occurs well into the plastic range--we can take advantage of the favorable plastic bending shape factor, Z/S, for tubes (ref. 17).

$$\frac{Z}{S} = \frac{4}{\pi}\left(1 + \frac{t}{D}\right) \tag{1.1}$$

Typical values for tubes listed in the AISC manual range from 1.30 up. About 96% of the fully plastic moment is developed at only twice yield strain. Thus, on the surface, the bending allowable of 0.75 F_y, corresponding to a shape factor of 1.25 seems quite reasonable, consistent with a bending allowable of 0.66 F_y for compact wide flange shapes. A difficult problem, however, lies in the definition of a D/t ratio below which members may be considered as compact.

Let us consider the range of behavior in bending for tubes with various D/t ratios, as shown in Figure 1.5 (ref. 18). For very stocky sections, we do not have to worry about local buckling. The moment-curvature (M-phi) behavior is fairly linear up to the yield moment. A modest amount of plastic curvature brings us to the fully plastic moment. With strain hardening, ultimate tensile failure is reached at a moment of about twice the yield moment, and at curvatures beyond the range of most practical applications.