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Acceptance of Steel and Aluminum Structures Designed per the Eurocodes at Fermilab

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Ad Hoc Alternative SBN / LBNF / DUNE Structural Codes and Standards Review Panel

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Abstract: This paper provides evidence that structures designed per EN 1990, EN 1991, EN 1993, EN 1999 (a subset of the "Eurocodes"), and EN 14620 have a generally equivalent level of safety to structures designed per the analogous US codes. Based on this evidence, the paper recommends accepting structures designed per these standards for use at Fermilab or Fermilab-operated space contingent on a review of the design documents for these structures.

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1.0 Introduction

Fermilab is expanding its leadership in global engineering design for future neutrino detectors, linear accelerators, and other scientific installations. In many instances, such projects are international and involve collaboration and partnership with many scientific institutions and universities from all over the world, including Europe. These institutions may contribute structures made of steel or aluminum for installation at Fermilab or Fermilab-operated space. These structures that are designed in Europe will conform to the Eurocodes, a set of building codes governing structural design.

Per Fermi Research Alliance's (FRA) contract with DOE, design of a structure at Fermilab is required to comply with the International Building Code (IBC), latest edition. IBC internally references other documents which govern different areas of structural design (loads on structures, structural steel design, etc.). These reference documents combined with IBC thus form the governing standards to which all structures at Fermilab (and elsewhere in the U.S.) must be designed. As of the publication of this paper, the latest edition of IBC is 2015.

Because structures are often designed and built for a single, unique purpose, each one must be individually designed for its intended use. A significant amount of work often goes into the design of a structure, and so to completely re-design or check a structure using a different set of standards would be extremely inefficient and a waste of resources. Therefore, some assurance is needed that a structure designed to the Eurocodes will not be immediately rejected for use at Fermilab. The purpose of this document is to establish whether structures designed to the Eurocodes will generally have a comparable or greater level of safety to those designed to the U.S. codes and can be accepted for use at Fermilab or Fermilab-operated space.

2.0 Work Group Members

The Mechanical Safety Subcommittee (MSS) formed an ad hoc panel comprised of the following structural engineers:

- Russ Alber, Facilities Engineering Services Section
- Brian Rubik, Facilities Engineering Services Section
- Arv Vasonis, Facilities Engineering Services Section

3.0 Goals

The goals for the ad hoc panel were:

Review alternative / international standards (listed below) and determine whether structures designed to these standards provide equivalent safety to structures designed to U.S. standards. This process is described in FESHM Chapter 2110.

a. EN 1990: Basis of structural design
b. EN 1991: Actions on structures

c. EN 1993: Design of steel structures
d. EN 1999: Design of aluminum structures
e. EN 14620: Design and manufacture of site built vertical, cylindrical, flat-bottomed steel tanks for the storage of refrigerated, liquefied gases with operating temperatures between 0 and -165C

Note that other topics commonly associated with structural design such as reinforced concrete and geotechnical / foundation design are not included in the scope of this paper. It is assumed that a design coming from Europe would be limited to the structure itself; any concrete or foundation design that would support the structure would be designed per US codes.

4.0 Findings

The goal of this document, as described in Section 3.0, is to determine whether structures designed to the Eurocodes and US codes provide equivalent levels of safety. Comparing every individual provision and formula would not be a straightforward process, would be prohibitively time-consuming, and is beyond the scope of this document. Rather, the method used for this review is to focus on several key concepts within structural design, gain an understanding of the overall approach the Eurocodes use, and compare this to the corresponding US code documents. A general conclusion can then be made as to whether a structure designed to the Eurocodes can provide a comparable level of safety as one designed to the US codes. Established as such and subject to review, Fermilab may then accept submission of structures designed to the Eurocodes.

4.1 Overall Document Comparison

In general, the selected Eurocode standards clearly correspond to an existing US standard in name and scope. Table 1 shows which Eurocode corresponds to which US code. The most significant difference is in EN 1990 *Basis of Structural Design*. The information in this document describes the fundamentals of structural design and is applicable to every structure regardless of materials used. By contrast, this information is divided (with some overlap) between IBC, ASCE 7, and the material-specific US codes. This topic is discussed further in Section 4.2 of this document.

Table 1 – Selected Eurocodes and corresponding US code documents

Eurocode	Corresponding U.S. Code
EN 1990	IBC, ASCE 7, AISC 360, ADM1
EN 1991	ASCE 7
EN 1993	AISC 360
EN 1999	ADM1
EN 14620	(Not a Structural Code)

After reviewing EN 14620, it was determined that any structural design in this standard is deferred to the relevant Eurocodes (EN 1990 through EN 1999). All other provisions in this standard are related to containment of liquefied gases and are outside of the expertise of this panel. This standard was therefore not reviewed in detail and compared to an analogous US standard.

Overall, the scope of the selected Eurocode documents matches the US code documents listed. Both sets of documents provide a consistent and safe approach to designing steel and aluminum structures.

4.2 Design Basis

The design basis is the underlying philosophy and principles that govern all structural design. This concept is very important to understanding levels of reliability and safety of structures designed to either the Eurocodes or US codes.

4.2.1 Reference Period

Structures are designed to withstand loads that are a statistical worst-case over a given reference period. Selecting a reference period dictates the loads that a structure is expected to resist, and so is directly related to the reliability of a structure.

In the Eurocodes, the reference period is discussed in EN 1990. Table B2 discusses how a design using all the applicable Eurocode documents results in a certain reliability using a 50-year reference period. Provision 4.1.2(7) also mentions that climatic actions (i.e. wind load) is based upon a 50-year reference period.

In the US codes, Table C.1.3.1a in ASCE 7 lists acceptable reliability for structures, all for a 50-year service period; the same as in the Eurocodes.

4.2.2 Reliability and Design Methodology

The reliability of a structure is its probability of failure over a given reference period; a structure with a higher reliability index (β) has a safer and more robust design. The way in which the design provisions achieve the target reliability index is the design methodology.

Table B2 of EN 1990 indicates that designing to the Eurocodes would "generally lead to a structure with a β value greater than 3.8 for a 50 year reference period." Table C.1.3.1a in ASCE 7 shows that for typical buildings (Risk Category II), reliability indexes of 3.0 to 4.0 for a 50-year service period are used. Note that the β value of 4.0 is for rare cases; review of literature including *Load and Resistance Factor Design for Steel*, the publication that introduced this concept for steel design in the US, indicates that a reliability index of 3.0 is generally targeted. Therefore, the reliability of structures designed to the Eurocodes appears to be generally equal to or greater than that of structures designed to the US codes.

The design methodology of the Eurocodes, as described in EN 1990, is to use the "partial factor method" to compare demand and capacity of a structure in various "limit states." The partial factor method is described in Section 6 of EN 1990 and uses partial factors (γ) that increase the demand on a structure and reduce the capacity of a structure in order to provide some buffer between nominal demand and capacity, or a factor of safety. The principles of limit state design are described in Section 3 of EN 1990. The general concept is that designers are required to check multiple limit states in addition to strength (stability, fatigue, etc.).

The design methodology of the US codes is very similar. Load and resistance factor design (LRFD) is used for design of most structures. In this method, load factors (γ) are applied to loads and resistance factors (φ) are applied to capacities as in the Eurocode method. A generalized version of either method is shown in the following equation:

$$[Resistance\ Factor]\times[Nominal\ Resistance] \geq \sum([Load\ Factors]\times[Nominal\ Loads])$$

The general idea behind the US code method and the Eurocode method is the same: provide a factor of safety between demand on a structure and its capacity in order to achieve a desired level of reliability. Like the Eurocodes, the US codes also require checking multiple limit states beyond strength. Comparison of specific load and resistance factors (partial factors) and limit states is discussed in the following sections.

4.2.3 Load and Resistance Factors / Partial Factors

The load and resistance factors or partial factors are the means by which the code documents achieve the desired level of structural reliability. As such, the factors must be looked at as a group rather than individually; variation in a factor on the demand can be compensated by an accompanying variation in a factor on the capacity. With that in mind, directly comparing Eurocode partial factors to US code load and resistance factors can still be informative.

Load Factors are applied to nominal loads as part of "load combinations," or several loads acting simultaneously on a structure. The load factors increase the nominal loads acting on a structure and are part of the overall factor of safety or reliability of the structure. A comparison of several common load combinations that include dead (permanent) load, live (variable) load, snow load and wind load is shown in Table 2. Some variation exists in the magnitude of the load factors as well as the combinations themselves, however the overall concept of combining simultaneous loads in various ways to yield the most critical combination for a given limit state is common in both sets of codes. Note that in Table 2 the US code terms (i.e. Dead Load, Live Load) are used for both cases for ease of comparison, rather than the Eurocode terms (i.e. Permanent Actions, Variable Actions).

Table 2 – Comparison of common load combinations in Eurocodes and US codes

Eurocode Load Combinations (Refer to Table A1.2(B) in EN 1990)	US Code Load Combinations (Refer to 2.3.2 in ASCE 7)
1.35D	1.4D
1.35D + 1.5L + 0.75S + 0.9W	1.2D + 1.6L + 0.5S
1.35D + 1.5S + 1.05L + 0.9W	1.2D + 1.6S + (1.0L or 0.5W)
1.35D + 1.5W + 1.05L + 0.75S	1.2D + 1.0W + 1.0L + 0.5S
1.0D + 1.5W	0.9D + 1.0W

Key: D = Dead Load L = Live Load
S = Snow Load W = Wind Load

Resistance factors are applied to the capacity of a structure, and therefore are related to the material used. Additionally, varying resistance factors are used for different failure modes in order to provide higher reliability to failure modes that may be more critical or dangerous. A comparison of common failure modes and their associated resistance factors in the Eurocodes and US codes for steel structures and aluminum structures is shown in Tables 3 and 4, respectively. As with the load combinations, there are minor differences in the magnitude of the reduction factors between the two sets of codes. The relative magnitudes do agree, however; for example, a brittle (more dangerous) failure mode such as tensile rupture has a greater strength reduction than a ductile (less dangerous) failure mode such as tensile yielding. The material failure modes checked in each code are also generally in agreement.

Table 3 – Comparison of resistance factors for common failure modes in steel design

Failure Mode	Eurocode Partial Factor (Refer to EN 1993)	US Code Resistance Factor (Refer to AISC 360)
Tensile Yielding	1 / 1.00 = 1.00	0.90
Tensile Rupture	1 / 1.25 = 0.80	0.75
Compression	1 / 1.00 = 1.00	0.90
Flexure	1 / 1.00 = 1.00	0.90
Shear	1 / 1.00 = 1.00	0.90 (most cases)
Weld Failure (multiple)	1 / 1.25 = 0.80	0.75 – 0.90
Bolt Failure – Tension / Shear	1 / 1.25 = 0.80	0.75
Bolt Failure – Slip	1 / 1.25 = 0.80 – 1 / 1.1 = 0.91	0.70 – 1.00

Table 4 – Comparison of resistance factors for common failure modes in aluminum design

Failure Mode	Eurocode Partial Factor (Refer to EN 1999)	US Code Resistance Factor (Refer to ADM1)
Tensile Yielding	1 / 1.10 = 0.91	0.90
Tensile Rupture	1 / 1.25 = 0.80	0.75
Compression	1 / 1.10 = 0.91 (most cases)	0.90
Flexure	1 / 1.10 = 0.91 (1 / 1.25 = 0.80 for rupture)	0.90 (0.75 for rupture)
Shear	1 / 1.10 = 0.91 (1 / 1.25 = 0.80 for rupture)	0.90 (0.75 for rupture)
Weld Failure	1 / 1.25 = 0.80	0.75
Bolt Failure – Tension / Shear	1 / 1.25 = 0.80	0.65 – 0.75
Bolt Failure – Slip	1 / 1.25 = 0.80 – 1 / 1.1 = 0.91	0.70 – 1.00

4.2.4 Limit States

A limit state is a failure criterion for a structure – note that failure can be anything that causes a structure to be unfit for its intended use. The Eurocodes divide these into two main categories: ultimate limit states and serviceability limit states. Ultimate limit states are defined as those associated with collapse or other similar forms of structural failure, while serviceability limit states are defined as those corresponding to conditions beyond which specified service requirements for a structure are no longer met.

Within EN 1990, Section 3.3 describes ultimate limit states and Section 6.4.1 lists those that must be checked. These limit states are:

6.4.1(1)a. EQU: loss of static equilibrium
6.4.1(1)b. STR: internal failure or excessive deformation (strength)
6.4.1(1)c. GEO: failure or excessive deformation of the ground
6.4.1(1)d. FAT: fatigue failure
6.4.1(1)e. UPL: uplift due to water pressure or other vertical actions
6.4.1(1)f. HYD: hydraulic heave

Items c, e, and f are geotechnical in nature and largely governed by EN 1997 and are therefore outside the scope of this document, although provisions for geotechnical failures are captured in the ASCE 7 load combinations. Item a corresponds to Chapter C of both AISC 360 and ADM1. Item b is covered by the load combinations in Chapter 2 of ASCE 7. Item d corresponds to Appendix 3 of both AISC 360 and ADM1. All ultimate limit states that are required to be considered by the Eurocodes are also required by the US codes, and vice-versa. A more detailed analysis of some specific strength limit states is shown in Section 4.5.

Sections 3.4 and 6.5 of EN 1990 describes the nature of serviceability limit states. Regarding specific design criteria, Section A1.4.2(2) states "The serviceability criteria should be specified for each project and agreed with the client" and "The serviceability criteria may be defined in the National Annex", meaning that serviceability requirements are project and/or country-specific. A "National Annex" is a supplementary document that can be issued by any country or jurisdiction wishing to adopt the Eurocodes to set certain region-specific parameters (i.e. ground snow loading or wind speed values). Therefore, no specific serviceability criteria are available for comparison to the US codes. However, the items listed for serviceability in the Eurocodes (appearance, occupant comfort, vibration, etc.) are the same criteria as used by the US codes.

Additionally, serviceability criteria that affect the functionality of a given structure would be independent of the code used to design the structure. Serviceability criteria also, by definition, do not impact the overall safety of a structure. It can therefore be concluded that the serviceability limit state of the Eurocodes and the US codes would not affect an equivalency determination for the purpose of this document.

4.3 Material / Section Properties

Provisions are written generically in both the Eurocodes and US codes to allow for the use of various section properties and steel / aluminum grades for structural components. However, certain material grades and structural sections are commonly used to make design and manufacturing more efficient. The following sections compare some of the most common material and section properties for steel and aluminum structures.

4.3.1 Steel Material Properties

While there are multiple grades of steel available, the mechanical properties are essentially constant among them all and are therefore prescribed by both the Eurocodes and US codes. Table 5 shows a comparison of the prescribed material properties for steel; the metric units are converted to imperial units for ease of comparison. The values of the properties listed in both sets of codes are close enough that any design differences would be negligible.

Table 5 – Comparison of prescribed steel mechanical properties

Steel Material Property	Eurocode Value	Eurocode Value (Imperial Units)	US Code Value
Young's Modulus (E)	210,000 N/mm^2 (Ref. EN1993-1-1 3.2.6(1))	30,500 ksi	29,000 ksi (Ref. AISC 360 Table B4.1)
Shear Modulus (G)	81,000 N/mm^2 (Ref. EN1993-1-1 3.2.6(1))	11,700 ksi	11,200 ksi (Ref. AISC 360 Section E4)
Poisson's Ratio (ν)	0.3 (Ref. EN1993-1-1 3.2.6(1))	0.3	0.3 (Ref. AISC 360 Comm. E7.1)
Coefficient of Thermal Expansion (α)	12 x 10^{-6} per K (Ref. EN1993-1-1 3.2.6(1))	6.7 x 10^{-6} per °F	7.8 x 10^{-6} per °F (Ref. AISC 360 App. 4: 4.2.3.1)

Several common steel grades are used in structural steel design. This list is not exhaustive; many other grades of steel are available and can be used per the Eurocodes and US code documents. Rather, the list examined here is meant as a general comparison between common design practices for the Eurocodes and US codes. Table 6 shows a comparison of common steel grades and their properties. Both sets of codes appear to use a lower strength and higher strength steel for both members and bolts which have similar strength values. The Eurocode documents treat welds differently in that the weld material is required to have a strength equal to or greater than the base material. This difference is not significant since weld material strength is almost never less than the base material strength in a design to the US codes.

Table 6 – Comparison of common steel grades

Eurocode Steel Grade[1]	Eurocode Value		Eurocode Value (Imperial Units)		US Steel Grade[1]	US Code Value	
	Yield (f_y)	Ultimate (f_u)	Yield (f_y)	Ultimate (f_u)		Yield (f_y)	Ultimate (f_u)
S235	235 N/mm^2	360 N/mm^2	34 ksi	52 ksi	A36	36 ksi	58 ksi
S355	355 N/mm^2	490 N/mm^2	51 ksi	71 ksi	A992	50 ksi	65 ksi
8.8	--	800 N/mm^2	--	116 ksi	A325	--	105 – 120 ksi
10.9	--	1000 N/mm^2	--	145 ksi	A490	--	150 ksi
Welds[2]	--	--	--	--	E70 Electrodes	--	70 ksi

Note 1: The steel grades presented in this table are for a comparison of strengths only; steel grades appearing on the same row (i.e. S235 and A36) are not necessarily equivalent in composition or other attributes.
Note 2: Strength of weld material is required to be equal to or greater than that of the base material, and is therefore not a separate design parameter (Ref. EN1993-1-8 Section 4.2(2)).

4.3.2 Aluminum Material Properties

As with steel, aluminum mechanical properties are mostly independent of grade and are prescribed by the code documents. Table 7 shows a comparison of the prescribed material properties for aluminum; the metric units are converted to imperial units for ease of comparison. The values of the properties listed in both sets of codes are close enough that any design differences would be negligible.

Table 7 – Comparison of prescribed aluminum mechanical properties

Steel Material Property	Eurocode Value	Eurocode Value (Imperial Units)	US Code Value
Young's Modulus (E)	70,000 N/mm^2 (Ref. EN1999-1-1 3.2.5(1))	10,200 ksi	10,100 ksi (Ref. ADM1 Table A.3.1)
Shear Modulus (G)	27,000 N/mm^2 (Ref. EN1999-1-1 3.2.5(1))	3,900 ksi	3,800 ksi (Ref. ADM1 Table A.3.1)
Poisson's Ratio (ν)	0.3 (Ref. EN1999-1-1 3.2.5(1))	0.3	0.33 (Ref. ADM1 Table A.3.1)
Coefficient of Thermal Expansion (α)	23 x 10^{-6} per K (Ref. EN1999-1-1 3.2.5(1))	13 x 10^{-6} per °F	13 x 10^{-6} per °F (Ref. ADM1 Table A.3.1)

Material properties of aluminum vary depending on alloy, temper, and material thickness. A particular type of aluminum is often selected based on many requirements beyond strength, and there are many types to choose from. As such, an in-depth comparison of aluminum grade properties is outside the scope of this document. However, it seems likely that for a given aluminum type designed to the Eurocodes, a type of aluminum with similar properties will be available in the US codes.

For reference, Tables 3.2a and 3.2b in EN 1999-1-1 and Table A.3.3 in ADM1 list nominal strengths for numerous aluminum types.

4.3.3 Typical Steel Sections

As with material properties, both sets of codes allow for the use of any structural steel shapes, however some common shapes are available and widely used in design. It's unlikely that exact dimensions of steel shapes will match between the Eurocodes and US codes, especially considering that two sets of units are used (metric vs. imperial). However, a more qualitative approach is used to compare the typical steel sections used in each set of codes. This level of comparison is appropriate because the method of calculating the capacity of a section (i.e. its failure modes) depends in part on the shape of the section.

Common steel shapes used in design according to the Eurocodes are shown in EN 1993-1-1 Tables 5.2 and 6.2. The general shape types seen are: I, U (or C), L, T, and box/tube-shaped. All of these shapes are also seen in the US codes, shown in AISC 360 Table B4.1a and B4.1b. It is therefore likely that similar failure modes are checked in the design of steel members to either the Eurocodes or the US codes. A more in-depth look into how each set of codes handles particular failure modes is shown in Section 4.5.

4.3.4 Typical Aluminum Sections

Although some common shapes of aluminum are used, designs using aluminum are often unique and therefore use a wide variety of specialized sections. The sets of codes allow for such variation by including provisions for analysis of each component of a section, regardless of overall shape.

Because of this variation, it is difficult to compare "typical" aluminum sections between the two code sets. Figure 1.1 in EN 1999-1-1 and Figures B.5.1 to B.5.4 in ADM1 show some examples of cross-sections that are possible with aluminum. Rather than attempt to compare the code sets in this regard, this document will focus on how each code analyzes the aluminum sections. More detail on this topic is given in Section 4.5.

4.4 Typical Loading

In the Eurocodes, loading on structures is divided into two main categories: permanent loads and imposed loads. Permanent loads include the self weight of the structure itself along with any other components that are fixed to the structure. Imposed loads are any other loading that will act on the structure throughout its design life, including loads from occupancy, wind load, snow load, seismic load, etc. Because of the nature of structures that will likely be part of this international collaboration program, this document will focus on how the Eurocodes treat permanent (dead) loads and loads due to occupancy (live load).

Permanent loads are treated similarly in both the Eurocodes and US codes; the weight of the structure and any other fixed components must be included in all relevant design cases. Both sets of codes offer recommended densities of common materials to aid in calculating the total permanent load on a structure. Tables showing these values are shown in EN 1991-1-1 Annex A and ASCE 7 Tables C3-1 and C3-2. These values should not be different since material density should be the same regardless of location – Table 8 compares densities recommended by both codes for a few materials and shows general agreement.

Table 8 – Comparison of recommended material densities

Material	Eurocode Value	Eurocode Value (Imperial Units)	US Code Value
Steel	77.0 – 78.5 kN/m^3	490 – 500 lb/ft^3	492 lb/ft^3
Aluminum	27.0 kN/m^3	172 lb/ft^3	170 lb/ft^3
Concrete (normal weight)	24.0 kN/m^3	153 lb/ft^3	150 lb/ft^3

To determine the live load on a structure, both sets of codes first classify the occupancy or use of the structure. Rather than compare recommended values, this document will compare how each code set treats live load. Because of the nature of structures that will likely be a part of the exchange program, the live load on a structure will have to be agreed upon by the designer and the owner / user of the structure instead of being dictated by a code document. For example, a platform grating that will only be accessed by a maintenance worker will have a different live load than a platform grating that is intended for large groups such as a tour even though the code may recommend a single value for a grating in an industrial application.

Both sets of codes define live load as load imparted on a structure by people, moveable objects (furniture, storage, moveable partitions, etc), vehicles, and similar cases. Some important design considerations for live load are:

1. Live loads may either be taken as uniformly distributed or concentrated loads, whichever is more demanding. Required by EN 1991-1-1 Section 6.1(2) and ASCE 7 Section 4.4.

2. Live loads may be distributed to some parts of the structure but not others in order to find the worst case for any component of the structure. Required by EN 1991-1-1 Section 6.2.1(1) and ASCE 7 Section 4.3.3.

3. For a live load acting over a large area, the total load may be reduced. Refer to EN 1991-1-1 Section 6.2.1(4) and ASCE 7 Section 4.7. The minimum area that is allowed to use this reduction is 10 m^2 (108 ft^2) in the Eurocodes and between 100 and 400 ft^2 in the US codes, depending on structural component type.

As illustrated by the above, the treatment of dead and live loads in both sets of codes is similar enough that a structure designed to either one would be required to resist a generally equivalent total loading.

4.5 Theory / Equations for Common Structural Components

As with other sections, comparison of how each code analyzes structural components will not be complete or exhaustive. Rather, a general comparison of common limit states for both steel and aluminum members and connections is presented which will allow for reasonable conclusions regarding equivalency.

For convenience, a table of symbols/definitions is shown in Table 9, including both Eurocode and US code notations. Note that this list is not exhaustive; for a complete listing and definition, see the code document in which the symbol appears.

Table 9 – Partial list of symbols and definitions

Eurocode Symbol	US Code Symbol	Definition
γ	φ	Resistance factor
γ	γ	Load factor
f_y	F_y	Material yield stress
f_u	F_u	Material ultimate stress
N	P	Design resistance – tension or compression
M	M	Design resistance – bending moment
F	R	Design resistance – bolts or welds
A	A	Area
W	S or Z	Section modulus

4.5.1 Tension – Steel

Per EN 1993-1-1 Section 6.2.3, the design tension resistance should be taken as the smaller of:

a) the design plastic resistance of the gross cross-section

$$N_{pl,Rd} = \frac{A f_y}{\gamma_{M0}} \tag{6.6}$$

b) the design ultimate resistance of the net cross-section at holes for fasteners

$$N_{u,Rd} = \frac{0,9 A_{net} f_u}{\gamma_{M2}} \tag{6.7}$$

As shown in the equations above, the design resistance is the lesser of tensile yielding on the gross cross section and tensile rupture on the net cross section (which includes effects of bolt holes and shear lag).

AISC 360 Section D2 lists the design tension resistance P_n as the smaller of:

(a) For tensile yielding in the gross section:

$$P_n = F_y A_g \tag{D2-1}$$

$$\phi_t = 0.90 \text{ (LRFD)} \qquad \Omega_t = 1.67 \text{ (ASD)}$$

(b) For tensile rupture in the net section:

$$P_n = F_u A_e \tag{D2-2}$$

$$\phi_t = 0.75 \text{ (LRFD)} \qquad \Omega_t = 2.00 \text{ (ASD)}$$

Similar to the Eurocode provisions, the US code calculates the design resistance as the lesser of tensile yielding on the gross cross section and tensile rupture on the net cross section. AISC 360 also includes effects of bolt holes and shear lag in the calculation of net section area.

4.5.2 Tension – Aluminum

EN 1999-1-1 Section 6.2.3 gives the design tension resistance as:

(2) The design tension resistance of the cross-section $N_{t,Rd}$ should be taken as the lesser of $N_{o,Rd}$ and $N_{u,Rd}$ where:

a) general yielding along the member: $$N_{o,Rd} = A_g f_o / \gamma_{M1} \qquad (6.18)$$

b) local failure at a section with holes: $$N_{u,Rd} = 0{,}9 A_{net} f_u / \gamma_{M2} \qquad (6.19a)$$

Note that design capacity at welds is omitted here, since it will be covered in a later section. The capacity of an aluminum section in tension is the lesser of tensile yielding of the gross cross section and tensile rupture of the net cross section. ADM1 shows similar provisions:

a) For tensile yielding in the gross section:

For unwelded members and members with transverse welds

$$P_{nt} = F_{ty} A_g \qquad (D.2\text{-}1)$$

b) For tensile rupture in the net section:

For unwelded members

$$P_{nt} = F_{tu} A_e / k_t \qquad (D.2\text{-}3)$$

Note that k_t is a factor that adjusts for the type of aluminum alloy, but is generally 1.0. The provisions for calculating the tensile capacity of an aluminum member are very similar in both the Eurocodes and US codes.

4.5.3 Compression – Steel

The limit state that often governs design of a steel member in compression is buckling. For simplicity, this section will compare how the Eurocodes and US codes treat buckling capacity of steel members only.

Section 6.3.1 in EN 1993-1-1 defines the buckling resistance of a compression member as:

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}} \quad \text{for Class 1, 2 and 3 cross-sections} \qquad (6.47)$$

$$N_{b,Rd} = \frac{\chi A_{eff} f_y}{\gamma_{M1}} \quad \text{for Class 4 cross-sections} \qquad (6.48)$$

Note that the “class” of a cross-section takes into account effects of local buckling (similarly accounted for in AISC 360), which is beyond the scope of this document. The buckling reduction factor, χ, is defined as:

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \quad \text{but } \chi \leq 1{,}0 \qquad (6.49)$$

where $\Phi = 0{,}5\left[1 + \alpha\left(\bar{\lambda} - 0{,}2\right) + \bar{\lambda}^2\right]$

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} \quad \text{for Class 1, 2 and 3 cross-sections}$$

$$\bar{\lambda} = \sqrt{\frac{A_{eff} f_y}{N_{cr}}} \quad \text{for Class 4 cross-sections}$$

The slenderness factor, λ-bar, includes the elastic critical buckling force of the member, N_{cr}. Substituting given values and solving for N_{cr} gives:

$$N_{cr} = \frac{\pi^2 EI}{L_{cr}^2},$$

which is Euler's critical buckling force. In AISC 360 Section E3, the compressive strength of a steel member is given as:

$$P_n = F_{cr} A_g \qquad \text{(E3-1)}$$

The *critical stress*, F_{cr}, is determined as follows:

(a) When $\frac{KL}{r} \leq 4.71\sqrt{\frac{E}{F_y}}$ (or $\frac{F_y}{F_e} \leq 2.25$)

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y \qquad \text{(E3-2)}$$

(b) When $\frac{KL}{r} > 4.71\sqrt{\frac{E}{F_y}}$ (or $\frac{F_y}{F_e} > 2.25$)

$$F_{cr} = 0.877 F_e \qquad \text{(E3-3)}$$

Note that both cases include the section's elastic buckling stress, defined as:

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} \qquad \text{(E3-4)}$$

This is the same as Euler's critical buckling force, written in terms of stress rather than force. Also note that in both sets of codes, both elastic (Euler) and inelastic (Johnson) buckling are covered similarly by the design equations. Elastic buckling would generally govern members with a higher slenderness, while inelastic buckling would govern members with a lower slenderness.

Although the approaches that the Eurocodes and US codes take to buckling in compression members is different, the effects that each consider are similar (i.e. slenderness, local buckling, Euler's critical buckling force/stress). It can therefore be concluded that the design procedure is based on sound engineering principles and the level of safety of a component designed to either code would be generally equivalent.

4.5.4 Compression – Aluminum

As with steel, the limit state that often governs design of aluminum members in compression is buckling. EN 1999-1-1 gives the design buckling resistance of a compression member as:

$$N_{b,Rd} = k\chi\omega_x A_{eff} f_o / \gamma_{M1} \quad (6.49a)$$

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} \quad \text{but } \chi \le 1,0 \quad (6.50)$$

where:

$$\phi = 0,5(1 + \alpha(\bar{\lambda} - \bar{\lambda}_0) + \bar{\lambda}^2)$$

$$\bar{\lambda} = \sqrt{\frac{A_{eff}\, f_o}{N_{cr}}} \quad \text{but for members with transverse welds, see 6.3.3.3(3)} \quad (6.51)$$

Note that these equations are similar to the equations for steel design, and that local buckling is also considered in these equations. The parameter λ-bar appears in these equations as well, which is related to the elastic critical buckling force of the member. ADM1 gives the buckling capacity of a member in compression as:

$$P_{nc} = F_c A_g \quad (E.2\text{-}1)$$

where

LIMIT STATE	F_c	λ
yielding	F_{cy}	$\lambda \le \lambda_1$
inelastic buckling	$(B_c - D_c\lambda)\left(0.85 + 0.15\dfrac{C_c - \lambda}{C_c - \lambda_1}\right)$	$\lambda_1 < \lambda < C_c$
elastic buckling	$\dfrac{0.85\pi^2 E}{\lambda^2}$	$\lambda \ge \lambda_2$

Note that local buckling is considered separately in Section E.3. Similar to the comparison for compression in steel members, the provisions in the Eurocodes and US codes look different, but the effects that each considers are the same (member global buckling, slenderness, and local buckling). Therefore, it can be concluded that the design methodologies for compression in aluminum members would generally lead to members with equivalent levels of safety.

4.5.5 Flexure – Steel

For design of steel sections in flexure, three limit states generally govern the overall capacity of a member: elastic/plastic yielding of the gross section, local buckling of section elements (i.e. flanges or webs), and global buckling of the member (for I-shaped sections, lateral torsional buckling is the governing global buckling mode).

In EN 1993-1-1 Section 6.2.5, the bending moment capacity of a section is given as:

(2) The design resistance for bending about one principal axis of a cross-section is determined as follows:

$$M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl}\ f_y}{\gamma_{M0}} \qquad \text{for class 1 or 2 cross sections} \qquad (6.13)$$

$$M_{c,Rd} = M_{el,Rd} = \frac{W_{el,min}\ f_y}{\gamma_{M0}} \qquad \text{for class 3 cross sections} \qquad (6.14)$$

$$M_{c,Rd} = \frac{W_{eff,min}\ f_y}{\gamma_{M0}} \qquad \text{for class 4 cross sections} \qquad (6.15)$$

Similar to design for compression, the "class" of a cross section takes into account section element slenderness and therefore local buckling effects. These design equations consider two of the three flexural limit states: elastic/plastic yielding of the section and local buckling effects. Design provisions for global buckling of the member is given in Section 6.3.2, where a reduction factor χ is used to account for buckling effects:

(3) The design buckling resistance moment of a laterally unrestrained beam should be taken as:

$$M_{b,Rd} = \chi_{LT} W_y \frac{f_y}{\gamma_{M1}} \qquad (6.55)$$

where W_y is the appropriate section modulus as follows:

- $W_y = W_{pl,y}$ for Class 1 or 2 cross-sections
- $W_y = W_{el,y}$ for Class 3 cross-sections
- $W_y = W_{eff,y}$ for Class 4 cross-sections

χ_{LT} is the reduction factor for lateral-torsional buckling.

(1) Unless otherwise specified, see 6.3.2.3, for bending members of constant cross-section, the value of χ_{LT} for the appropriate non-dimensional slenderness $\bar{\lambda}_{LT}$, should be determined from:

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \quad \text{but } \chi_{LT} \leq 1{,}0 \qquad (6.56)$$

where $\Phi_{LT} = 0{,}5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0{,}2\right) + \bar{\lambda}_{LT}^2\right]$

α_{LT} is an imperfection factor

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}}$$

M_{cr} is the elastic critical moment for lateral-torsional buckling

In these provisions, global buckling of the member is considered while including effects of local buckling of section elements.

Similar to compression, AISC 360 considers the same limit states but using a different approach than the Eurocodes. Members are categorized by section type (I-shaped, tubes, etc.) and slenderness of their section elements. Once a member is given a category, all applicable limit states are checked. For example, in Section F2, I-shaped sections with "compact" section elements (no local buckling is possible) includes provisions for checking elastic/plastic section yielding and global buckling of the member:

1. Yielding

$$M_n = M_p = F_y Z_x \quad \text{(F2-1)}$$

where

F_y = *specified minimum yield stress* of the type of steel being used, ksi (MPa)

Z_x = plastic section modulus about the x-axis, in.^{3} (mm^3)

2. Lateral-Torsional Buckling

(a) When $L_b \le L_p$, the *limit state* of *lateral-torsional buckling* does not apply.

(b) When $L_p < L_b \le L_r$

$$M_n = C_b\left[M_p - \left(M_p - 0.7F_yS_x\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p \quad \text{(F2-2)}$$

(c) When $L_b > L_r$

$$M_n = F_{cr}S_x \le M_p \quad \text{(F2-3)}$$

In Section F3, I-shaped members that include section elements (flanges / webs) that can have local buckling effects are checked for this additional limit state:

2. Compression Flange Local Buckling

(a) For sections with noncompact flanges

$$M_n = M_p - \left(M_p - 0.7F_yS_x\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \quad \text{(F3-1)}$$

(b) For sections with slender flanges

$$M_n = \frac{0.9Ek_cS_x}{\lambda^2} \quad \text{(F3-2)}$$

Although the organization and methodology of the design provisions of the Eurocodes and US codes are different, the limit states that are checked are the same (elastic/plastic section yielding, local buckling and member global buckling). As with compression, it can be concluded that a member designed to either set of codes would have a generally equivalent level of safety.

4.5.6 Flexure – Aluminum

As with steel design, three limit states often govern design of aluminum members in flexure: elastic/plastic yielding of the section, member global buckling and element local buckling. EN 1999-1-1 Section 6.2.5 gives the design resistance of a section as:

(2) The design resistance for bending about one principal axis of a cross section M_{Rd} is determined as the lesser of $M_{\mathrm{u,Rd}}$ and A1 $M_{\mathrm{o,Rd}}$ A1 where:

$M_{\mathrm{u,Rd}} = W_{\mathrm{net}} f_{\mathrm{u}} / \gamma_{\mathrm{M2}}$ in a net section and A2 (6.24a) A2

A2 $M_{\mathrm{u,Rd}} = W_{\mathrm{u,eff}} f_{\mathrm{u}} / \gamma_{\mathrm{M2}}$ in section with transverse weld (6.24b) A2

A1 $M_{\mathrm{o,Rd}} = \alpha W_{\mathrm{el}} f_o / \gamma_{\mathrm{M1}}$ at each cross-section (6.25) A1

The first equation accounts for elastic/plastic yielding of the section and the third is for element local buckling (α is as shape factor that varies according to section class, which depends on element slenderness). Section 6.3.2 gives member buckling resistance as:

$$M_{b,Rd} = \chi_{LT}\omega_{xLT}\alpha W_{el,y} f_o / \gamma_{M1} \quad (6.55a)$$

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \quad \text{but } \chi_{LT} \le 1 \quad (6.56)$$

The buckling capacity includes member slenderness parameter λ-bar.

ADM1 Section F.2 gives the flexural yielding strength as:

For the limit state of yielding, the nominal flexural strength M_{np} of wrought products is the least of $Z\,F_{cy}$, $1.5S_t\,F_{ty}$, and $1.5S_c\,F_{cy}$

For the limit state of yielding, the nominal flexural strength M_{np} of cast products is the lesser of $S_t\,F_{ty}$ and $S_c\,F_{cy}$.

Local buckling is accounted for in Section F.3 as:

$$M_{nlb} = F_b\,S_{xc} \quad \text{(F.3-2)}$$

Note that F_b includes effects of local element slenderness given in section B.5. Member capacity for the lateral-torsional buckling mode is given in Section F.4:

For the limit state of lateral-torsional buckling, the nominal flexural strength M_{nmb} is:

LIMIT STATE	M_{nmb}	SLENDERNESS LIMITS
inelastic buckling	$M_{np}\left(1-\frac{\lambda}{C_c}\right)+\frac{\pi^2 E\lambda S_{xc}}{C_c^3}$	$\lambda < C_c$
elastic buckling	$\pi^2 E S_{xc}/\lambda^2$	$\lambda \ge C_c$

As in the Eurocode provisions, member slenderness is accounted for in the slenderness parameter λ.

Similar to the design provisions for steel members, the Eurocodes and US codes use different approaches in the design of aluminum members in flexure. However, the same limit states are considered (elastic/plastic yielding of the section, member global buckling and element local buckling). Given this, aluminum members in flexure designed to either the Eurocodes or US codes would have generally equivalent levels of safety.

4.5.7 Bolts – Steel

The select limit states that will be compared are bolt tension, shear, and combined tension and shear. These are some of the most common limit states that often govern design of bolted steel connections.

EN 1993-1-8 Table 3.4 gives equations for the design resistance of bolts in tension, shear, and combined shear and tension. Tensile capacity of a bolt is given as:

Tension resistance [2)]	$F_{t,Rd} = \dfrac{k_2 \; f_{ub} \; A_s}{\gamma_{M2}}$

In this equation, k_2 is a reduction factor for countersunk bolts; therefore the bolt capacity is essentially the ultimate stress of the bolt multiplied by its area. Shear capacity of a bolt is given as:

Shear resistance per shear plane	$F_{v,Rd} = \dfrac{\alpha_v \; f_{ub} \; A}{\gamma_{M2}}$

As shown, the shear capacity is the ultimate stress of the bolt multiplied by its area as well as the number of shear planes (α_v). In the US codes, the design capacity of bolts in either shear or tension is shown in AISC 360 Section J3.6:

$$R_n = F_n A_b \qquad \text{(J3-1)}$$

Similar to the design provisions in the Eurocodes, the capacity of a bolt in shear or tension is the bolt ultimate stress multiplied by its area. For shear capacity, the bolt area is multiplied by the number of shear planes (discussed in the AISC 360 code commentary).

For combined shear and tension in a bolt, EN 1993-1-8 Table 3.4 gives the design equation as:

Combined shear and tension	$\dfrac{F_{v,Ed}}{F_{v,Rd}} + \dfrac{F_{t,Ed}}{1{,}4\,F_{t,Rd}} \leq 1{,}0$

And the design provision for a bolt in combined shear and tension in AISC 360 is:

$$R_n = F'_{nt} A_b \qquad \text{(J3-2)}$$

$$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{rv} \leq F_{nt} \quad \text{(LRFD)} \qquad \text{(J3-3a)}$$

In both sets of codes, the design provision combines shear and tensile stress by effectively reducing the allowable tension in a bolt based on the magnitude of shear.

As shown, the selected design provisions for bolts are very similar in the Eurocodes and the US codes.

4.5.8 Bolts – Aluminum

Similar to the section on bolt design for steel structures, this section will consider bolt tension, shear, and combined shear and tension. EN 1999-1-1 gives the design resistance of a bolt in tension as:

Tension resistance $$F_{t,Rd} = \frac{k_2 \; f_{ub} \; A_s}{\gamma_{M2}} \quad (8.17)$$

In this equation, k_2 is a reduction factor for material and countersunk bolts; therefore the bolt capacity is essentially the ultimate stress of the bolt multiplied by its area (including any reduction for bolt threads). Shear capacity of a bolt is given as:

Shear resistance per shear plane: $$F_{v,Rd} = \frac{\alpha_v \; f_{ub} \; A}{\gamma_{M2}} \quad (8.9)$$

As shown, the shear capacity is the ultimate stress of the bolt multiplied by its area as well as a factor accounting for material and inclusion/exclusion of threads (α_v). In the US codes, capacity of a bolt in tension is given as:

$$R_n = (\pi(D - 1.191/n)^2/4)F_{tu} \quad \text{(J.3-1)}$$

Capacity of a bolt in shear with threads included (Eq. J.3-2) or excluded (Eq. J.3-3) in the shear plane is given as:

$$R_n = (\pi(D - 1.191/n)^2/4)F_{su} \quad \text{(J.3-2)}$$

$$R_n = (\pi D^2/4)F_{su} \quad \text{(J.3-3)}$$

Similar to the design provisions in the Eurocodes, the capacity of a bolt in shear or tension is the bolt ultimate stress multiplied by its area, with a reduction in area used for bolt threads.

For combined shear and tension, EN 1999-1-1 lists the design capacity as:

Combined shear and tension $$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1{,}4\,F_{t,Rd}} \leq 1{,}0 \quad (8.20)$$

This provision is identical to that used in EN1993-1-8, discussed above. In ADM1, the provision for combined shear and tension points to AISC360:

$$R_n = F_n A_b \quad \text{(J.3-6)}$$

where
F_n is F_{nt} or F_{nv} determined in accordance with the *Specification for Structural Steel Buildings (ANSI/AISC 360).*

In both cases, design of bolts in both shear and tension is identical to the provisions given in the steel design codes. It can therefore be concluded that the procedure for the design of aluminum bolts in both sets of codes would produce structures with equivalent levels of safety.

4.5.9 Welds – Steel

For simplicity, this document will consider design of fillet welds. Although there are many types of welds and weld processes, fillet welds are very common and a comparison of this weld type will provide a sound basis for equivalency while allowing for a reasonable scope of review.

Design capacity of a fillet weld is given in EN 1993-1-8 Section 4.5.3:

(2) Independent of the orientation of the weld throat plane to the applied force, the design resistance per unit length $F_{w,Rd}$ should be determined from:

$$F_{w,Rd} = f_{vw.d}\, a \qquad \text{... (4.3)}$$

(3) The design shear strength $f_{vw.d}$ of the weld should be determined from:

$$f_{vw.d} = \frac{f_u / \sqrt{3}}{\beta_w \gamma_{M2}} \qquad \text{... (4.4)}$$

The design capacity of the weld is its unit strength multiplied by the effective throat and the weld length. The unit strength is the base metal ultimate strength divided by $\sqrt{3}$ and a correlation factor β_w. Filler metal is required by EN 1993-1-8 Section 4.2.2 to be of equivalent or greater strength than the parent material:

(2) The specified yield strength, ultimate tensile strength, elongation at failure and minimum Charpy V-notch energy value of the filler metal, should be equivalent to, or better than that specified for the parent material.

The factor β_w is to account for filler metal that has a higher ultimate stress as compared to "lower" strength steels (i.e. S235 and S355).

The design capacity of a fillet weld in AISC 360 is given in section J2.4:

$$R_n = F_{nw} A_{we} \qquad \text{(J2-3)}$$

Load Type and Direction Relative to Weld Axis	**Pertinent Metal**	**ϕ and Ω**	**Nominal Stress (F_{nBM} or F_{nw}) ksi (MPa)**
FILLET WELDS INCLUDING FILLETS IN HOLES AND			
Shear	Base	Governed by	
	Weld	$\phi = 0.75$ $\Omega = 2.00$	$0.60F_{EXX}$ [d]

The weld capacity is therefore the nominal stress ($0.60F_{EXX}$) multiplied by the effective weld area, which is defined as the effective throat multiplied by the weld length. Because 0.60 is the rounded form of $1/\sqrt{3}$, both sets of codes use very similar approaches to determining the capacity of a fillet weld. The US code does not include the β_w factor because the strength of the filler metal is directly included in the design equations rather than being considered through a parent material "correlation factor".

Note that other topics related to welds, such as weld processes, welder qualifications, weld inspection, etc., is beyond the scope of this document and will be discussed in detail in a subsequent white paper.

4.5.10 Welds – Aluminum

Similar to the section above on design of welds in steel structures, this section will limit the comparison of design of welds in aluminum structures to fillet welds loaded parallel to the direction of the weld (loaded in shear) for simplicity.

EN1999-1-1 Section 8.6.3 gives the required effective throat thickness of a double fillet welded joint loaded parallel to the weld axis as:

$$a \geq \sqrt{\frac{2}{3}} \frac{\tau_{Ed}\, t}{f_w / \gamma_{Mw}} \qquad (8.36)$$

For ease of comparison to the US codes, this equation can be re-written to show the capacity of a single fillet weld:

$$Weld\ Capacity = \frac{ahf_w}{2 \times \sqrt{2/3}}$$

Note that the partial factor (discussed in Section 4.2.3) was omitted for clarity. As shown, the Eurocodes give the weld capacity as the unit strength of the weld (f_w) multiplied by the weld area (ah) and a coefficient. In this case, the coefficient is 0.612. In the US codes, ADM1 Section J2.5 gives the strength of a fillet weld in shear as:

shear	Base	$0.6F_{tuw}$	$S_w L_{we}$
	Weld 1	$0.6(0.85F_{tuw})$	$S_{we} L_{we}$

The weld capacity is again defined as the unit strength of the weld (F_{tuw}) multiplied by the weld area ($S_{we}L_{we}$) and a coefficient. In the US codes, this coefficient is 0.51. The main difference appears to be in the 0.85 factor, which according to the code commentary in ADM1 accounts for lower filler shear strengths compared to wrought alloys as determined through testing. Regardless of the reason for its inclusion in ADM1, this difference would result in a weld designed in an aluminum structure to US codes to have approximately 0.83 times the capacity of a weld designed to the Eurocodes.

While this is a noteworthy difference between the sets of codes, the basic weld capacity calculation is very similar and relies on the same governing principles. The treatment of weld capacity in the Eurocodes appears to be rigorous and based on sound engineering principles, and so this difference is not cause for outright rejection of an equivalency between the sets of codes. Weld capacity in aluminum structures will, however, be flagged as a topic that will be reviewed with extra diligence given this finding.

4.6 Selected Research Paper Review

Several research papers written on the topic of code comparisons were reviewed and their findings are summarized in this section. The scope of these papers is often very limited since an exhaustive comparison between sets of codes would be a monumental task.

In *Comparison of Eurocode EC3 and American AISC 360 to the Design of Large Span Structures*, several aspects of design were considered (i.e. material properties, tension design, compression design). Eurocode 3 and AISC 360 were compared in this paper, although the focus was on large span structures. In Section 4 on cross section classification, one finding is that "…it can be seen that there is no drastic difference between the limits in both codes". In Section 6 on tension design, one finding is "AISC-360 and EC3 both consider tensile yielding in the gross section and tensile rupture in the net section as the two primary limit states for tension members". One of the conclusions in Section 10 of the paper is "both codes estimate ratio of slenderness the same way…". While this paper discusses some of the minor differences found between the two codes, the overall conclusion seems to be that the methods used and resulting designs are generally very similar.

In *Comparison between Eurocodes and North American and Main International Codes for Design of Bolted Connections in Steel Bridges*, several sets of codes were reviewed regarding design of bolted connections in steel bridges, including both the Eurocodes and US codes. Some of the aspects considered in this paper are the overall design method, geometric considerations, and strength limit states. Although many small differences between the sets of codes are discussed, one of the final conclusions of the paper is "Eurocode seemed to be the most conservative for the typical case studied in terms of shear, bearing, and combined shear and tension resistance".

In *Comparative Study of Major International Standards*, several sets of codes were compared with regards to wind loading. Although outside of the scope of this document, wind load provision comparison can provide another data point in the overall goal of Eurocode – US code equivalency. As with the other research papers, an in-depth look at the differences between design methods is presented. The overall conclusion of the paper, however, is that "while significant discrepancies are apparent in the comparison of the intermediary parameters, the overall loads are reasonably consistent."

As seen in these select research papers, the overall conclusion seems to be that although the Eurocodes and US codes approach some aspects of design in different ways, the resulting design is generally equivalent regardless of which code set is used.

4.7 Structural Engineer Qualification / Drawing and Calculation Requirements

In the United States, structural drawings and calculations are typically "stamped" by a licensed engineer in the state where the structure will be located. In most states, a Professional Engineer (PE) license is sufficient, but some states (including Illinois) require a Structural Engineer (SE) license. The requirements for an individual to acquire either license are graduation from an accredited university engineering program, pass the Fundamentals of Engineering (FE) exam, practice engineering for a given amount of time under a licensed PE or SE (typically four years), and pass either a day-long (PE) or two day-long (SE) exam demonstrating expertise in the field. The main purpose of requiring a license to practice structural engineering is to ensure the safety of the public.

Fermilab's contract with DOE stipulates that structural design will comply with the requirements of the IBC, but there are no explicit requirements in the contract for stamping structural drawings and calculations or for engineers to be licensed. Additionally, Fermilab has no internal Authority Having Jurisdiction (AHJ) or building code official governing structural design work.

Although there are no explicit license requirements governing structural work within FRA's contract with DOE, several federal regulatory documents indicate that using licensed engineers for contracted work is required. For work performed by a subcontracted architecture or engineering firm, Federal Acquisition Regulation (FAR) clauses 52.236-25 explicitly require architects and engineers preparing designs for the federal government to be registered. Department of Energy Acquisition Regulation (DEAR) clause 970.5244-1 requires any architecture or engineering work subcontracted by the lab to be performed by licensed individuals. Additionally, Section 107 of the IBC states that "the construction documents shall be prepared by a registered design professional where required by the statutes of the jurisdiction in which the project is to be constructed". In Fermilab's case, the statutes of the jurisdiction do not explicitly require licensed professionals; however, this requirement is common practice for nearly every project in the US. Based on this and the above referenced federal regulations, it's reasonable to conclude that an engineer licensed to practice structural engineering in the jurisdiction in which a structure will be built should perform or review the construction documents and calculations for structures built at Fermilab or Fermilab-operated space.

In Europe, it appears that there is no requirement for drawings or calculations to be "stamped" or certified by an individual engineer. In some cases (i.e. for important structures), it may be required to have an independent check of the design before construction, but there does not appear to be an equivalent process in the US to certify that the design of a structure is in conformance with all applicable building codes. Additionally, engineers are qualified to practice in their field of expertise upon graduating from university and no certification similar to the PE or SE exists in Europe.

Because there are no requirements for licensed engineers or stamped documents for structural design work at Fermilab, the requirements appear to be equivalent between Europe and the special case at Fermilab. Although not required, an independent check by an engineer licensed to practice structural engineering of any structural design coming from Europe would be prudent and bring the process in line with best practice in the US as well as Europe. This independent check is discussed further in Section 5 of this document.

5.0 Recommendations

Based on the evidence presented in this document, the panel recommends that structures designed to the Eurocodes be allowed for use at Fermilab or Fermilab-operated space under the following conditions:

1. All drawings, specifications, calculations, and other design documents applicable for the structure are provided for review by an engineer licensed to practice structural engineering in the jurisdiction in which the structure will be built. Note that this may be a PE or SE, as required by the state.

2. A design basis for the structure that includes a list of reference codes / standards is provided.
3. Sufficient time is allotted for review of these documents which will depend on the length and level of detail of the documents and complexity of the structure.
4. Welded aluminum structures have been identified as an area in which the Eurocodes and US codes differ, and therefore special attention will be given to any structures that include welded aluminum parts.

Given these items, the documents will proceed through the standard review process as described in the Fermilab Engineering Manual. The scope of the review will be for general conformance to the applicable codes and to ensure that the design is based on sound engineering principles. The level of review will vary depending on the structure and documents received, but the goal will remain the same: to ensure a level of safety in the structure that meets or exceeds what is required by the governing codes listed in Fermi Research Alliance's contract with the Department of Energy.

6.0 References

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14. Fermi Research Alliance contract with the US Department of Energy
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