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NUMERICAL SIMULATION AND PARAMETRIC DESIGN OF STEEL RIGID CONNECTIONS BUILT WITH W SHAPES: STUDY AND ANALYSIS FOR QUALIFIED AND NON-QUALIFIED MODELS WITH LOW-CARBON UNION PLATES UNDER ANSI/AISC-360-16 STANDARDS.

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“Si el hombre simplemente se sentara y pensara en su fin inmediato y en su horrible insignificancia y soledad en el cosmos, seguramente se volvería loco, o sucumbiría a un entumecedor o soporífero sentido de inutilidad. Porque, podría preguntarse:

¿Por qué debería molestarme en escribir una gran sinfonía o luchar para ganarme la vida, o incluso amar a otro, cuando no soy más que un microbio momentáneo en una mota de polvo dando vueltas por la inmensidad inimaginable del espacio?”

—Stanley Kubrick.

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1 Abstract

Currently, using the orthodox and traditional analytical methods to calculate a steel connection it is almost impossible to determinate the stresses distribution under the forces that are subjected to it, and still it results also harder to have a depiction of the structural performance on its geometry. In this research work, there will be displayed an overview of the structural performance of steel connections using W sections as structural member shapes studying multiple configurations restricted under several parameters. Besides, it exposes a contrast between a conventional (*qualified*) prototype with respect to a configuration commonly built nowadays but with no punctual research about it, everything under the Specification for Structural Steel Buildings requirements of the American Institute of Steel Construction (*ANSI/AISC-360-16*). This research project is mainly focused on configurations with welded joints and union plates with low-carbon content as ASTM A36 and ASTM A572 gr. 50 just for saying.

To build and compute the solution, the approaching was based on the numerical technique known as Finite Element Method (FEM), which is employed for solving problems in computational mechanics and engineering problems. This case represents an exclusive application to solids mechanics problems for structural analysis field. The advantage of studying this models lies in that better conditions could be recreated to simulate a kind of computational environment where the terms assumed and belonging for each problem can give us a result more possibly adjusted to the real expected behaviour. Otherwise, another advantage is to compute and the discretization of complex domains complicated geometries.

There is a strong belief about the employment of numerical solutions and the computational resources in engineering, that consequently represents a step to the future of solving coupled complex problems, such as the nature of this research. In this thesis is also discussed the study of a possible optimization according to a further discussed methodology for design, mainly oriented to back up the unqualified configuration as potential solution in steel construction.

The objectives will be lately shown, but, principally it corresponds of providing of real constructive configurations optimizing the material used and creating models that could satisfy the strength and security required to guarantee a correct structural performance and implementation for edifications under particular conditions during their useful life.

2 Introduction

2.1 Historical Background of Steel Structures

The structural steel is one of the basic materials used in buildings and construction due to its properties and strength along the whole human civilizations history. The first used of steel structure in construction of countries can be traced back to the end of 18th century in British. Metal as structural material began with cast iron, used on a 30m arch span which was built in England in 1777-1779. The structure created was the Coalbrookdale Bridge over the River Severn which represents a turning point in history, because it changed the course of the Industrial revolution by introducing iron as structural material. After that, a number of cast iron bridges were built during the period 1780-1820 and it was also used for chain links on suspension bridges until about 1840 [2].

Soon after 1840, the wrought iron began replacing the cast iron and the most notorious building with this material it was the Brittania Bridge over Manai Straits in Wales during 1846-1850. This was a tubular girder bridge having spans of (70-140-140-70)m, and it was made with wrought-iron plates and angles. The term wrought iron refers to iron with a very low carbon content ($\leq 0.15\%$) while iron with a high content ($\geq 2\%$) is known as cast iron. The development of the Bessemer process and subsequent advances, such as open-hearth process, permitted the manufacture of steel at competitive prices. Since 1890, steel has replaced wrought iron as the principal metallic building material [2].

2.2 Philosophies of Design

There are two philosophies in current use:

- The working stress design (referred to by AISC as Allowable Stress Design).
- Limit states design (referred to by AISC as Load and Resistance Factor Design).

The Allowable Stress Design (ASD) has been the principal philosophy over 100 years. During the last 20 years or so, structural steel design has gone toward a more rational and probabilistic design procedure referred to as “limit states” [2]. The limit states design includes methods often known as:

- Ultimate strength design.
- Strength design.
- Plastic design.
- Load factor design.
- Limit design.

Besides, in structural analysis it must be considered the possibility of an overload which provides a reserve strength beyond service loads already contemplated. Overloads may arise from changing the use of a particular structure for it was designed, from underestimation of effects of loads by wrong or over simplifications in structural analysis, and even by construction procedures. According to reality, the fact of an understrength existent in the steel members, bolts, welds and other materials might be possible; and it must be also considered deviations between nominal and real dimensions that will be fit within statistically possible limits and tolerances.

No matter what philosophy of design is employed, the fundamental argument is to provide for an adequate safety and strength enough. The study of what constitutes the suitable formulation of structural safety has been going forward during the past thirty years. The main trust has been to examine by several probabilistic methods the chances of failure presents in a member, connection or system.

Limit states can be splitted into two categories: strength and serviceability. The first one consists of such behavioural phenomena as achieving ductile maximum stress, buckling, fatigue, sliding,

etcetera. Serviceability limit states are concerned with occupancy of the building, such as deflections, vibration, permanent deformation and cracking.
The current approach to a simplified method for obtaining a probability-based assessment for structural safety uses *first-order second-moment reliability methods*. Such methods assume the load effect **S** and the resistance **R** are random variables. When the the resistance **R** exceeds the effect load **S** there will be a margin of safety. Otherwise, if **R** exceeds **S** by a large amount, there will be some probability that **R** may be less than **S**.
The failure (achievement of a limit state) may be examined comparing **S** with **R** (Figure 1), or in a logarithmic curve observing variation of $\ln(\frac{R}{S})$ represented by **M** (Figure 1), failure can be described and represented by the section where p_f is pointed. The distance between the failure line and the mean value of the function **M** is defined as a multiple β of the standard deviation σ of the function. Such multiple β is also known as *reliability index*. As the value of β grows also the margin of safety does [2].

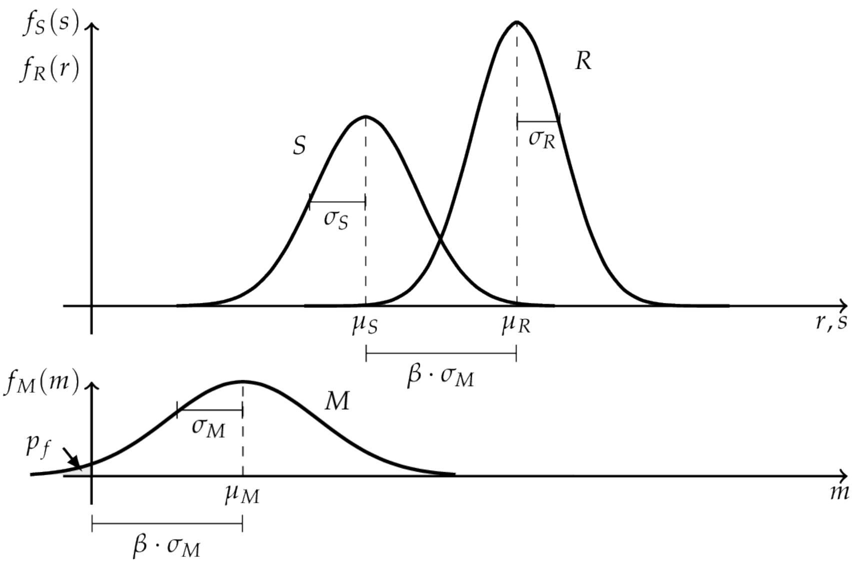


Figure 1: Frequency distributions of load S and resistance R (top) and reliability index β (bottom), (*Performance Goals of Roadway Bridges of Cost Action, Ademovic Naida and others*).

The reliability index provides useful information such as:

- It can give an indication of the consistency of safety for various components and systems using traditional design methods.
- It can be used to establish new methods which will have consistent margins of safety.
- It can be employed to vary in a rational manner the margins of safety for those components and systems having a greater or lesser need for safety than that required in ordinary situations.
  In general terms, the expression to back up structural safety requirement may be written as

$$\phi \cdot R_n \geq \sum \gamma_i \cdot S_i \tag{1}$$

where the left side represents the resistance of the component or system, and the right side represents the load expected to be subjected. On the strength side, the value of nominal resistance R_n is multiplied by a resistance factor ϕ to obtain the design strength. On the load side of equation are the various load effects such as the loads that may inflict in the structure. These loads are multiplied by overload factors and grouped together in load combinations to get the factored loads applied over the structure [2].

2.3 Steel and Properties

2.3.1 Introduction

The structural performance of any building under loads applied is reflected by the materials used in edification. Since introduction of steel as construction material until about 1960, the steel employed was identified as a carbon steel by American Society for Testing and Materials (ASTM) with designation A7 ($f_y = 2310 kg/cm^2$), and the structural designers referred to it just as steel without any other further attributes or properties. Other structural steels, such as special corrosion resistant low alloy steel (A242) and a more easily weldable steel (A343), were available but rarely used in construction but principally used in bridge design [2].
Currently, there are many types of structural steels available that allow the designer the use of increased strength material in higher stress regions rather than increasing the dimensions of members. Besides, the designer must anticipate the weather conditions will be inflicted on members and considering constructive details like the facility of welding with other elements. This is because certain steels provide better weldability respect to others; some are more suitable than others for pressure vessels, either at temperatures well above or below room temperatures.
The main structural shapes used for construction are identified as hot-rolled members and may be classified as carbon steels, high-strength low-alloy steels and alloy steels. The general requirements for these are stipulated under ASTM A6/A6M specification. Structural steels are referred to by ASTM designations, and also by many proprietary names. For design purposes, the *yield stress* in tension is the parameter to establish an allowable stress.

2.3.2 Carbon Steels

Carbon steels are divided into four categories depending on percentage carbon content.

Designation	**Percentage content**
Low carbon	less than 0.15%
Mild carbon	(0.15-0.29)%
Medium carbon	(0.30-0.59)%
High carbon	(0.60-1.70)%

Structural carbon steels are in the mild carbon category; a steel such as A36 (Figure 2: green curve) has maximum carbon content varying from 0.25 to 0.29% depending on thickness. As higher is carbon percent raises the yield stress but it reduces ductility, making welding process more difficult [2].

2.3.3 High-strength Low-alloy Steels

This category includes steels with yield stresses from 40 to 70ksi (2800-4900kg/cm^2) exhibiting the well-defined yield-point shown in purple curve (Figure 2). The difference to carbon steels lies in addition of small quantities of alloy elements such as chromium, columbium, copper, manganese, molybdenum, nickel, phosphorus, vanadium or zirconium that improves mechanical properties and resistance against atmospheric corrosion regarding to other steels available. Actually those steels are low-carbon and have an amount up 2.0% of Manganese [1].
Applications of high-strength low-alloy steels(HSLA) include oil and gas pipelines, heavy-duty highway and off-road vehicles, construction and farm machinery, industrial equipment, storage tanks, mine and rail road cars, barges and dredges, snowmobiles, lawn mowers, and passenger car components. Bridges, offshore structures, power transmission towers, light poles, and building beams and panels are additional uses of these steels [9].
HSLA can be divided into six categories:

- Weathering steels, which are improved for atmospheric corrosion resistance and solid-solution strengthening.

- Microalloyed ferrite-pearlite steels, which contain very small amounts (usually less than 0.10%) of strong carbide or carbonitrideforming elements such as nobium, vanadium, and/or titanium for precipitation strengthening and possibly transformation temperature control.
- As-rolled pearlitic steels, these may include carbon-manganese but which may also have small additions of other alloying elements to enhance strength, toughness, formability, and weldability.
- Acicular ferrite (low-carbon bainite) steels, which are low-carbon (less than 0.05% C) with an excellent combination of high yield strengths, (as high as $7000 kg/cm^2$, or 100ksi) weldability, formability, and good toughness.
- Dual-phase steels, which have a micro structure of martensite dispersed in a ferritic matrix and provide a good combination of ductility and high tensile strength.
- Inclusion-shape-controlled steels, which provide improved ductility and through-thickness toughness by the small additions of calcium, zirconium, or titanium, or perhaps rare earth elements so that the shape of the sulphide inclusions is changed from elongated stringers to small, dispersed, almost spherical globules.

2.3.4 Alloy Steels

These steels are weldable with proper procedures, and ordinarily do not require additional heat treatment after they have been welded. Low-alloy steels may be quenched and tempered to obtain yield strengths of 80 to 100ksi ($5600 - 7000 kg/cm^2$). Yield strength is often defined by the 0.20% offset strain, since these steels do not show an equable yield stress [2].
The heat treatment consists of rapid cooling with water or oil from at least $950°C$ to about $150 - 200°C$; then tempering again by reheating to at least $620°C$ and allowing to cool. Tempering, even though process reduces the strength and hardness somewhat from the quenched material, greatly improves the ductility [2].

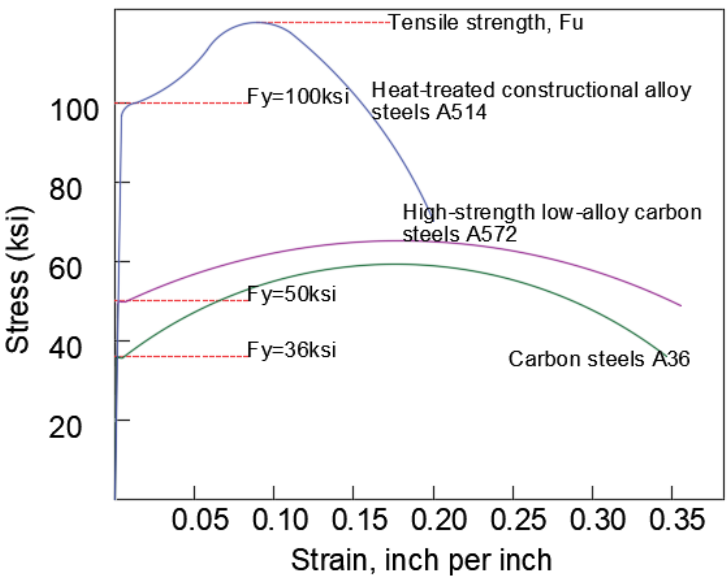


Figure 2: Typical stress-strain curves.

2.3.5 ASTM Designations

All steels listed are approved under the AISC Specifications. For hot-rolled structural shapes, plates, and bars, such tests shall be made in accordance with ASTM A6/A6M; for sheets, such tests shall be made in accordance with ASTM A568/A568M; for tubing and pipe, such tests shall be made in accordance with the requirements of the applicable ASTM standards listed above for those product forms [10].

(a)Hot-rolled structural shapes	
ASTM A36/A36M ASTM A529/A529M ASTM A572/A572M ASTM A588/A588M	ASTM A709/A709M ASTM A913/A913M ASTM A992/A992M ASTM A1043/ASTM A1043M
(b)Hollow Structural Sections (HSS)	
ASTM A53/A53M Grade B ASTM A500/A500M ASTM A501/A501M ASTM A618/A618M	ASTM A847/A847M ASTM A1065/A1065M ASTM A1085/A1085M
(c)Plates	
ASTM A36/A36M ASTM A242/A242M ASTM A283/A283M ASTM A514/A514M ASTM A529/A529M	ASTM A572/A572M ASTM A588/A588M ASTM A709/A709M ASTM A1043/A1043M ASTM A1066/A1066M
(d)Bars	
ASTM A36/A36M ASTM A529/A529M	ASTM A572/A572M ASTM A709/A709M
(e)Sheets	
ASTM A606/A606M	ASTM A1011/A1011M SS, HSLAS, AND HSLAS-F

2.4 Tension Members

2.4.1 Introduction

Tension members are structural elements axially loaded by direct forces that tend to enlarge their length. An element under these conditions is subjected by an uniform stress distributed in a given cross section [1]. They can be present as principal structural members in bridge and roof trusses, in truss structures such as transmission towers, and seismic and wind bracing systems in high buildings. Tension members may consist of a single shape or built up from a number of it and others.
Any structural shape may be used as tension member as long as end configuration in members owes strength enough to support the mechanic elements applied on such point. A scheme (Figure 3) is shown with different types of cross-section commonly used in edifications.

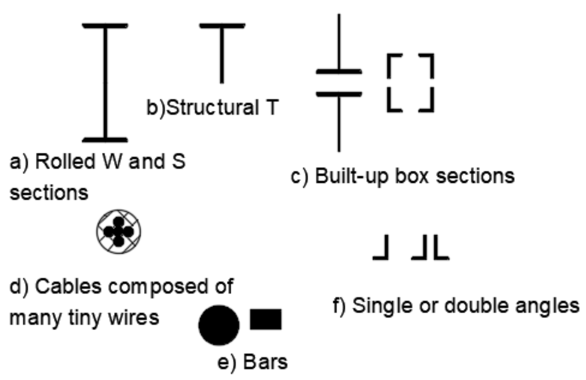


Figure 3: Cross-section of typical tension members.

2.4.2 Net Area

As a result for holes provided at the connection, the member cross-sectional area is reduced and thus it inflicts on strength depending on the size and location of the holes [1]. The net area A_n of a member is the sum of the products of the thickness and the net width of each element computed as follows: In computing net area for tension and shear, the width of a bolt hole shall be taken as $1/16in$ greater than the nominal dimension of the hole.
For a chain of holes extending across a part in any diagonal or zigzag line, the net width of the part shall be obtained by deducting from the gross width the sum of the diameters or slot dimensions, of all holes in the chain, and adding, for each gage space in the chain, the quantity

$$\frac{s^2}{4g^2}$$

Where:

- s = longitudinal center-to-center spacing of any two consecutive holes.
- g = transverse center-to-center spacing between fastener gage lines.

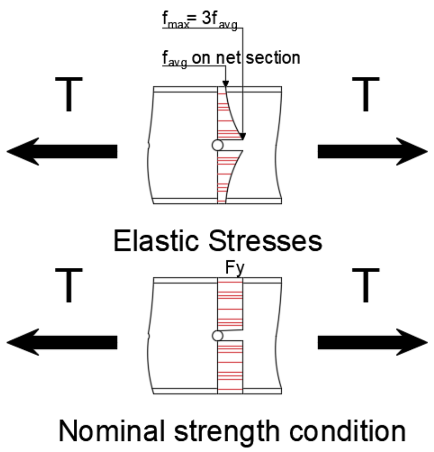


Figure 4: Stress distribution with holes present.

2.4.3 Effective Area

The effective net area for tension members shall be determined as follows:

1. When a tension load is transmitted directly to each of the cross-sectional elements by fasteners or welds, the effective net area A_e is equal to the net area A_n.
2. When a tension load is transmitted by bolts or rivets through some but not all of the cross-sectional elements of the member, the effective net area A_e shall be computed as

$$A_e = U \cdot A \tag{2}$$

Where:

- A = area defined below.
- U = reduction coefficient.

$$U = 1 - \frac{\bar{x}}{L} \leq 0.90 \tag{3}$$

- $\bar{x}$ = connection eccentricity.
- L = length of connection in the directions of loading.

Larger values of U are permitted to be used when justified by tests or other rational criteria.

a. When the tension load is transmitted only by bolts or rivets:
$A = A_n$

b. When the tension load is transmitted only by longitudinal welds to other than a plate member or by longitudinal welds in combination with transverse welds:
$A = A_g$

c. When the tension load is transmitted only by transverse welds:
$A = A_{con}$ (area of directly connected elements)
$U = 1.0$

d. When the tension load is transmitted to a plate by longitudinal welds along both edges at the end of the plate for $l \geq w$:
$A = A_g$

For:

1) $l \geq 2w$ .... $U = 1.00$
2) $2w > l \geq 1.5w$ .... $U = 0.87$
3) $1.5w > l \geq w$ .... $U = 0.75$

Where:

- l = length of weld.
- w = plate width (distance between welds).

2.4.4 Lag Shear Effect

Stress distribution is uniform regions away from connection, this is in body member. However, for a tension members different from plates, frequently the end connection is just supported by some section elements. The shear lag effect reduces effectiveness member components which are not directly connected either a gusset plate or another anchor plate, and therefore, it is reflected in reducing strength design of member [1]. The shear lag effect must be considered for bolted and welded joints in tension members.

2.4.5 Nominal Strength

The strength of tension member is described in terms of limit states that govern. Generally, tension members are connected in ends to gusset plates and joined by bolts or weld. When a bolted configuration is elected, it must be considered the cross-section reduced for bolts. Having said this, the controlling strength limit state may be one of the following [1]:

- **Yielding of the gross section**.
  A steel member axially loaded can resist a greater force than yield load without reaching rupture. However, big elongation resultant due to yield in member without control continuing to adjacent elements failure. It is important to restrict elongation and consequently the yield in gross section.

- **Fracture of the net area**.
  For tension members having holes, such as for bolts, the reduced cross-section is referred to as the net area. Holes in members cause stress concentrations at service load (Figure 5). Before yielding in member occurs, the end connection region may suffer strain hardening and the fracture may be present near to this region.

- **Block shear fracture**.
  In union plate, if the group of bolts are closer each other, a material rectangle block can be teared apart from member. This mode of failure is also known as *shear block failure*.

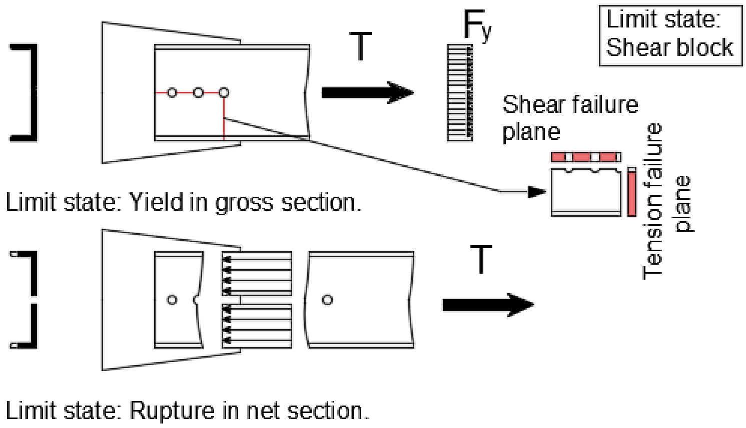


Figure 5: Limit states in tension members.

For yielding in the gross section

$$P_n = F_y \cdot A_g \tag{4}$$

Where:

- F_y = specified minimum yield stress.
- A_g = gross cross sectional area.
- $\phi_t = 0.90$ (LRFD), $\Omega_t = 1.67$ (ASD).

For fracture in net section

$$P_n = F_u \cdot A_e \tag{5}$$

Where:

- F_u = specified minimum tensile stress.
- A_e = effective net area.
- $\phi_t = 0.75$ (LRFD), $\Omega_t = 2.00$ (ASD).

Minimum Limit Values for Reduction Coefficient U

Previous issues of this Specification have presented values for U for bolted or riveted connections of W, M, and S shapes, tees cut from these shapes, and other shapes. These values are acceptable for use in lieu of calculated values from Equation 5 and are retained here for the convenience of designers.
For bolted or riveted connections the following values of U may be used:

- W, M, or S shapes with flange widths not less than two-thirds the depth, and structural tees cut from these shapes, provided the connection is to the flanges and has no fewer than three fasteners per line in the direction of stress:
  $U = 0.90$
- W, M, or S shapes not meeting the conditions of previous subparagraph, structural tees cut from these shapes, and all other shapes including built-up cross sections, provided the connection has no fewer than three fasteners per line in the direction of stress:
  $U = 0.85$

- All members having only two fasteners per line in the direction of stress:
  $U = 0.75$

When a tension load is transmitted by fillet welds to some but not all elements of a cross section, the weld strength will control.

2.5 Compression Members

2.5.1 Introduction

A column is a structural member used to transmit a compression load along a straight path of its longitudinal axis. Steel columns may be divided into three categories [1]:

- Short columns. The element is called this way when member length is the same order as total dimensions of its cross-section. In case of short columns, the load limit is that which causes material yielding and its capacity is not affected by any instability.
- Large columns. This category comprises elements with a length greater than its cross-section. The failure consists of element buckling and it is reached by critical load or buckling load (P_{cr}) even though the present stress is not high enough as yield stress.
- Intermediate columns. These type of columns also may fail by buckling, but the difference is that some strings reach yield stress respect to others. For this reason, this behaviour is known as inelastic.

2.5.2 Failure Modes

There are three general modes by which axially loaded members can reach failure. These modes of buckling are enlisted and briefly defined below [1]:

- **Flexural buckling**.
  Members are subjected to flexure, or bending, when they become unstable (also known as *Euler buckling*).
- **Local buckling**.
  It occurs when some parts of the cross-section of a column are so thin that they buckle locally in compression before other modes of buckling can occur. The susceptibility of a column to local buckling is measured by the width-thickness ratio of the parts of its cross-section.
- **Flexural torsional buckling**.
  This case may be presented in columns that have certain cross-sectional configurations. These columns fail either by twisting (torsion) along its longitudinal axis or by a combination of torsional and flexural buckling.

The tendency of a member to buckle is usually measured by its *slenderness ratio*, which is defined as the ratio of the length of the member to its radius of gyration.

2.5.3 Euler Elastic Buckling

Deriving differential equation for buckling of a column hinged in both ends subjected to bending is formulated under the next premises [1]:

- The member is totally prismatic and its cross-section has double symmetry.
- The column is straight before being loaded.
- The compression load is applied along centroidal axis of column.
- There are not transversal loads.
- Both ends of member are ideally pinned.

- Material is homogeneous and it obeys Hooke's law.
- Flat sections before deformation remain flat after deformation.
- The caused deformations are infinitesimal.
- There is not twisting in section.

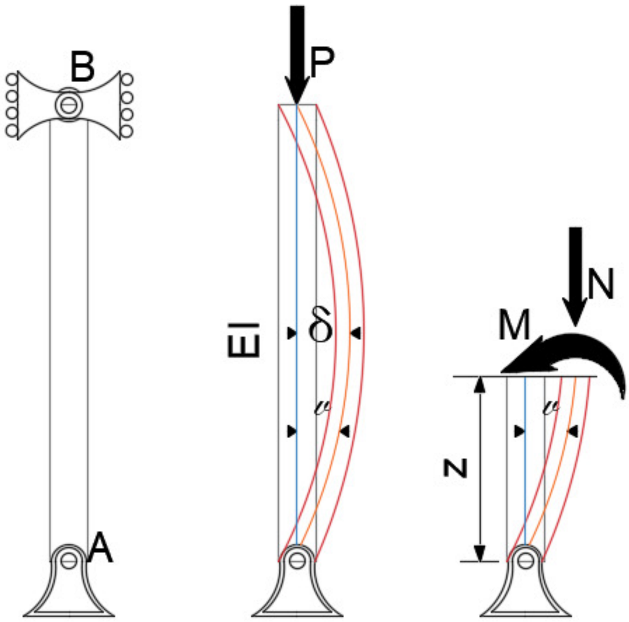


Figure 6: Column axially loaded and pinned at both ends.

At any location of z, the bending moment M_z on the member bent slightly is:

$$M_z = P \cdot v \tag{6}$$

and since

$$\frac{d^2v}{dz^2} = -\frac{M_z}{EI} \tag{7}$$

the differential equation to establish equilibrium along member results

$$\frac{d^2v}{dz^2} + \frac{P}{EI}v = 0 \tag{8}$$

replacing

$$k^2 = \frac{P}{EI}$$

the solution of this second-order differential equation may be expressed

$$v = A \sin kz + B \cos kz \tag{9}$$

Applying boundary conditions

$$v = 0 \ at \ z = 0; \ v = 0 \ at \ z = L$$

Replacing $v = 0$ at $z = 0$, the bent curve becomes

$$v = A \sin kz \tag{10}$$

And satisfaction of Equation 10 may be accomplished in three different ways:

- $A = 0$, and it means there is not deflection.
- $kL = 0$, what it means there is not applied force.
- $kL = N\pi$, the requirement for buckling to occur.

Thus

$$\left(\frac{N\pi}{L}\right)^2 = \frac{P}{EI}$$

The fundamental buckling mode, a single curvature deflection will occur when $N = 1$; thus the Euler critical load column is

$$P_{cr} = \frac{\pi^2 EI}{L^2} \tag{11}$$

2.5.4 Residual Stresses

Residual stresses and their distribution are very important factors affecting strength of axially loaded columns. The influence is noticeable in particular for columns with slenderness ratio varying from 40 to 120, a range that includes a very large percentage of practical columns [3]. A major cause of residual stress is the uneven cooling of shapes after hot-rolling. For instance, in a W shape the outer tips of the flanges and the middle of the web cool quickly, while areas that intersect web and flanges cool more slowly.
The quicker cooling parts of the sections, when solidified, resist further shortening, while those parts that are still hot tend to shorten further as they cool. The net result is that cooled more quickly have residual compressive stresses, and the slower cooling areas have residual tensile stresses. The magnitude of such residual stresses varies from about 10 to 15ksi ($700 - 1050 kg/cm^2$), although some values greater than 20ksi ($1400 kg/cm^2$) have been found [3].
As a column load is increased, some parts of column will quickly reach yield stress and go into the plastic regime because of residual stresses. The stiffness of column will be reduced and become a function of the part of cross-section that is still elastic. The buckling calculations for a particular column with residual stresses can be either handled by using effective moment of inertia I_e or by using the tangent modulus [3].
In columns, welding process can produce severe residual stresses that actually may approach

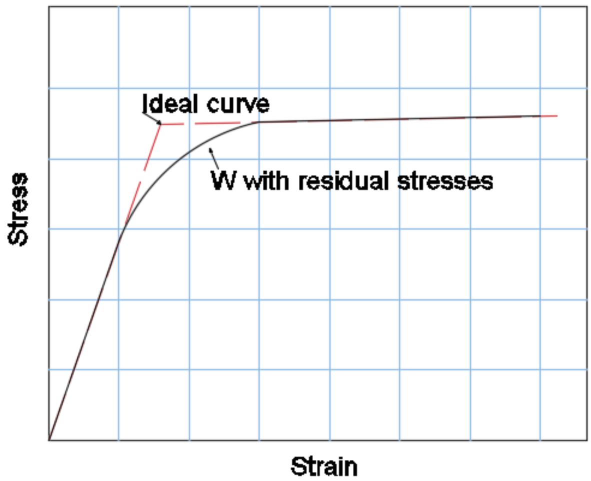


Figure 7: Effect on stress-strain relationship due to residual stresses.

the yield point in the vicinity of the weld. It is also important the fact that columns may be appreciably bent by the welding process, which is reflected affecting their load-carrying ability [3]. Residual stresses may also be caused during fabrication process when cambering is performed by cold bending, and due to cooling after welding. Cambering is the bending of a member in direction opposite to the direction of bending that will be caused after application of service loads [3].

2.5.5 Nominal Strength

There are two equations governing column strength, based on the limit state of flexural buckling, one for inelastic buckling which it is

$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) F_y \quad used\ for\ \frac{L_c}{r} \leq 4.71\sqrt{\frac{E}{F_y}} \tag{12}$$

and the other expression for elastic (Euler) buckling is given by

$$F_{cr} = 0.877 \cdot F_e \quad when\ \frac{L_c}{r} > 4.71\sqrt{\frac{E}{F_y}} \tag{13}$$

Where:

- F_{cr} = design stress for compression.
- F_y = specified minimum yield stress.
- F_e = elastic buckling stress determined according to equation 14.

$$F_e = \frac{\pi^2 E}{\left(\frac{L_c}{r}\right)^2} \tag{14}$$

Where:

- E = modulus of elasticity.
- $L_c = KL$ = effective length of member.
  - K = effective length factor dependent on support conditions.
  - L = laterally unbraced length of member.
- r = critical radius of gyration about plane of buckling.

Given these expressions and finally proceeding to compute design load for compression using

$$P_n = F_{cr} \cdot A_g \tag{15}$$

Where:

- A_g = gross area.
- $\phi_c = 0.90$ (LRFD), $\Omega_c = 1.67$ (ASD).

2.6 Shear on Rolled Beams

2.6.1 Introduction

As bending increases in member, shear stresses occur because of the changes in length of its longitudinal fibers. Considering a positive bending, the lower fibers tend to stretch (under tension stresses) and the upper fibers are shortened (due to compression). Regarding to this variation in deformations, the fibers has the tendency to slip over each other [3].
For the purposes of this research, the direction will be focused on all effects inflicted on the use of W shapes for beam-column connections.

2.6.2 Possible Cases where Contribution of Shear is Significant

Usually, shear is not a problem in steel beams, because the webs of rolled shapes are capable of resisting relatively "large" shearing forces. Then, below is shown some common situations where the contribution of shear might be really excessive [3]:

- Should large concentrated loads be placed near beam supports, therefore, they will cause large internal forces without increasing the bending moments. A clear example of this case of loading occurs in tall buildings where, on a particular floor, the upper columns are offset with respect to the columns located below. The loads from upper columns applied to beams of the level in question will be quite large if there are many stories above.
- Probably the most common shear problem occurs where two members (as a beam and a column) are rigidly connected together so that their webs lie in a common plane. This situation often occurs at the junction of beam and columns in rigid frame structures.
- Where beams are coped (Figure 8), shear can be a problem. In this particular case, the shear must be transferred to the remaining beam depth. Similar situations are presented where holes are cut in beams for ductwork or another applications.
- Theoretically, very heavily loaded short beams can have excessive shears, but practically, this does not occur frequently unless it is like the first case.
- Shear might be a great problem when shapes with very thin webs are employed, as in plate girders or in light-gage cold-formed steel members.

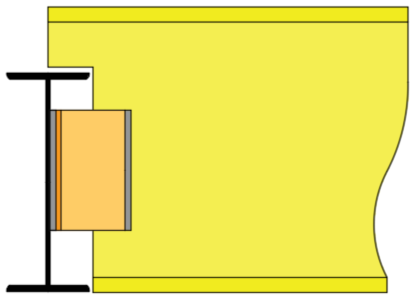


Figure 8: Beam coped.

2.6.3 Shear Stress Equation for Symmetrical Sections

Let us consider a thin slice dx from a beam shown as a free body (Figure 9). If a unit shear stress τ at a section y_1 from the neutral axis is desired, it is known that:

$$dC = \tau \cdot b\ dx \tag{16}$$

the horizontal forces arising from bending moment are:

$$C + dC = \int_{y_1}^{y_2} (f + df)\ dA \tag{17}$$

subtracting,

$$df = \frac{dM}{I} y \tag{18}$$

replacing equation 18 into equation 16, and solving for shear stress τ gives

$$\tau = \frac{dM}{dx} \left(\frac{1}{Ib} \right) \int_{y_1}^{y_2} y\ dA \tag{19}$$

where

$$Q = \int_{y_1}^{y_2} y\, dA \quad and \quad V = \frac{dM}{dx}$$

and finally, the familiar equation

$$\tau = \frac{VQ}{Ib} \tag{20}$$

For unusual procedures of steel design, the shear stress is computed as the average value over the gross area of the web neglecting the presence of any hole over it, thus [2]

$$\tau_{avg} = \frac{V}{A_w} = \frac{V}{t_w d} \tag{21}$$

Notice that for large holes cut in a beam to allow passage of pipes and ducts require special considerations and their effect cannot be neglected. To summarize, these assumptions for the shear stress equation for symmetrical cross-sections will help later to understand and computing solution in a more accurate way [2].

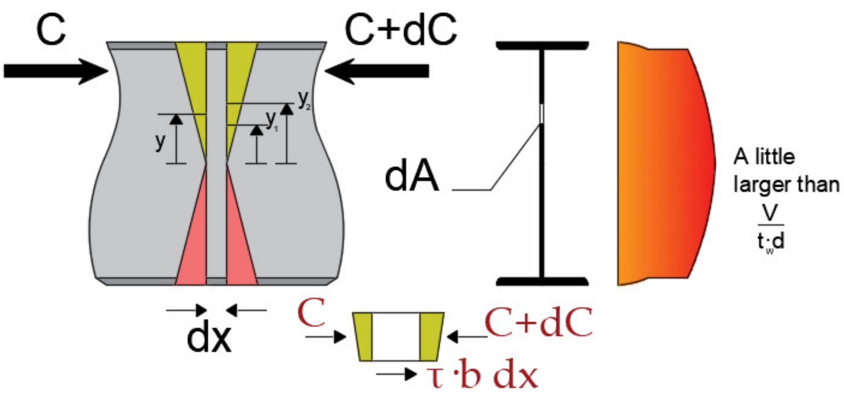


Figure 9: Flexural stresses involved in derivation of τ.

2.6.4 Nominal Strength

The shear strength expressions are shown in AISC Specification (chapter G).

I-Shaped Members and Channels (Shear strength of webs without tension field action)

The nominal shear strength, V_n, will be:

$$V_n = 0.6F_y \cdot A_w \cdot C_{v1} \tag{22}$$

where

- F_y = specified minimum yield stress of the steel being used.
- A_w = area of the web.
- C_{v1} = coefficient for shear strength. This parameter varies for different $\frac{h}{t_w}$ ratios , depending on whether shear failures would be plastic, inelastic, or elastic. Where:
  - h = clear distance between flanges less the fillet at each flange.
  - t_w = thickness of the web.

The value of C_{v1} and other parameters will be taken as follows:

- $C_{v1} = 1.00$, $\phi_v = 1.00$ (LRFD), $\Omega_v = 1.50$ (ASD) for

$$\frac{h}{t_w} \leq 2.24\sqrt{\frac{E}{F_y}}$$

- For the other I-shaped rolled members and channels, the value of resistance factors will be $\phi_v = 0.90$ (LRFD), $\Omega_v = 1.67$ (ASD); but C_{v1} will be considered regarding to the following cases:

  i. $C_{v1} = 1.00$ when

$$\frac{h}{t_w} \leq 1.10\sqrt{\frac{k_v E}{F_y}}$$

  ii. Otherwise

$$C_{v1} = 1.10\left(\frac{t_w}{h}\right)\sqrt{\frac{k_v E}{F_y}} \tag{23}$$

  The web plate shear buckling coefficient , k_v, is determined as follows:

  i. For webs without transverse stiffeners

$$k_v = 5.34 \tag{24}$$

  ii. For webs with transverse stiffeners

$$\begin{aligned} k_v &= 5 + \frac{5h^2}{a^2} \qquad (25)\\ &= 5.34 \;\; when \; \frac{a}{h} > 3.00 \end{aligned}$$

  where

  – a = clear distance between transverse stiffeners.

I-Shaped Members and Channels (Shear strength of interior web panels considering tension field action)

The nominal shear strength, V_n, will be (the following considerations are given for $\frac{a}{h} \leq 3.00$):

1. When

$$\frac{h}{t_w} \leq 1.10\sqrt{\frac{k_v E}{F_y}}$$

it is accomplished, then, the nominal shear strength is given by

$$V_n = 0.6F_y \cdot A_w \cdot C_{v2} \tag{26}$$

2. If such ratio does not satisfy the expression, then

$$V_n = 0.6F_y \cdot A_w \left[C_{v2} + \frac{1 - C_{v2}}{1.15\sqrt{1 + \frac{a^2}{h^2}}}\right] \tag{27}$$

subjected to:

$$\begin{aligned} \frac{2A_w}{A_{fc} + A_{ft}} &\leq 2.50 \\ \frac{h}{b_{fc}} &\leq 6.00 \\ \frac{h}{b_{ft}} &\leq 6.00 \end{aligned}$$

otherwise

$$V_n = 0.6F_y \cdot A_w \left[C_{v2} + \frac{1 - C_{v2}}{1.15 \left[\frac{a}{h} + \sqrt{1 + \frac{a^2}{h^2}} \right]} \right] \tag{28}$$

The value of C_{v2} and other parameters will be taken as follows:

- $C_{v2} = 1.00$, $\phi_v = 0.90$ (LRFD), $\Omega_v = 1.67$ (ASD) when

$$\frac{h}{t_w} \leq 1.10\sqrt{\frac{k_v E}{F_y}}$$

- $\phi_v = 0.90$ (LRFD), $\Omega_v = 1.67$ (ASD) when

$$1.10\sqrt{\frac{k_v E}{F_y}} < \frac{h}{t_w} \leq 1.37\sqrt{\frac{k_v E}{F_y}}$$

  then

$$C_{v2} = 1.10 \left(\frac{t_w}{h} \right) \sqrt{\frac{k_v E}{F_y}} \tag{29}$$

- $\phi_v = 0.90$ (LRFD), $\Omega_v = 1.67$ (ASD) and greater values for $\frac{h}{t_w}$ than

$$\frac{h}{t_w} > 1.37\sqrt{\frac{k_v E}{F_y}}$$

  then

$$C_{v2} = \frac{1.51 k_v E t_w^2}{h^2 F_y} \tag{30}$$

  where:

  - A_{fc} = area of compression flange.
  - A_{ft} = area of tension flange.
  - b_{fc} = width of compression flange.
  - b_{ft} = width of tension flange.
  - k_v = is the web plate shear buckling coefficient.

Single Angle and Tees

According to AISC Specification G3, the nominal shear strength of a single-angle leg or a tee stem is:

$$V_n = 0.6F_y \cdot b \cdot t \cdot C_{v2} \tag{31}$$

where:

- C_{v2} = web shear buckling strength coefficient, with $\frac{h}{t_w}$ equivalent to $\frac{b}{t}$ and $k_v = 1.20$.
- b = width of the leg resisting the shear force or the depth of the tee stem.
- t = thickness of angle leg or tee stem.

3 Steel Connections

3.1 Introduction

In general terms, a connection will be defined as set of union elements intersected at a node with the purpose of transmitting some stresses regarding to the idealized model conditions for its structuring. Frequently, the connections are joined either by weld or bolts.
In most cases, it is necessary the use of stiffening elements to provide of continuity in beam-column connections. It is also common the use of plates for reinforcement in some areas of the connection with the purpose of avoiding local failures. The principal interest of studying steel connections lies on the complexity of certain geometries and oversimplified assumptions for designing, and respectively, it is complicated to get additional information or knowledge from real panorama.

3.2 LRFD Classification

There are three types of steel connections identified [2]:

- Fully restrained. Also known as continuous frame or rigid frame, this configuration is assumed if the angle conservation is on order of 90% or more before service loads. Such connections are designated as “Type 1” in ASD or “FR” in LRFD.
- Simple framing. Also unrestrained or free-ended, this type of connection assumes that rotational restraint at the end of members is a little amount as a neglected term. For beams, this situation is just intended to transfer shear at the ends. Simple framing is usually assumed if the fact of angle conservation after load application is less than 20%. Structures using simple framing connections have long been called “Type 2” in ASD and are known as “Type PR” in LRFD.
- Partially restrained. This category includes al connections which rotational restrained is less than 90% and greater than 20%. This case is considered because in regard to a stiffness always existent. This type is not commonly used in ASD because of difficulty of obtaining its moment-rotation relationship for a given connection. the denomination of this connection in ASD is stipulated as “Type 3” and “Type PR” in LRFD.

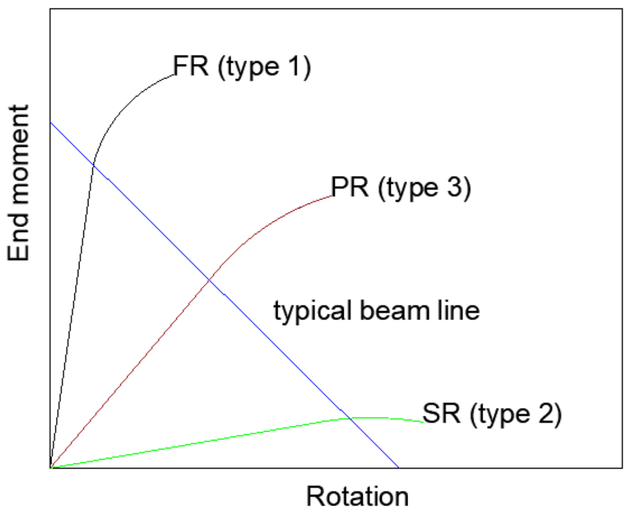


Figure 10: Scheme LRFD/ASD of steel connections classification.

3.3 Bolts

3.3.1 Advantages of High-strength Bolts

The great success and acceptance of using bolted connections with high-strength bolts are the following [3]:

- Smaller crews are involved, compared to riveting. The result is quicker steel construction erection.
- Compared respect to rivets, fewer bolts are required to provide the same strength.
- Good bolted joints can be made by people with a great deal less training and experience that is necessary to produce welded or riveted connections of equal quality. The proper installation of high-strength bolts can be learned within few hours.
- Cheaper equipment is employed to make bolted connections.

3.3.2 Types of High-strength Bolts

Use of high-strength bolts in *Specification for Structural Steel Buildings* are grouped according to material strength as follows:

- Group A: ASTM F3125/F3125M Grades A325, A325M, F1852 and ASTM A354 Grade BC
- Group B: ASTM F3125/F3125M Grades A490, A490M, F2280 and ASTM A354 Grade BD
- Group C: ASTM F3043 and F3111

The use of Group C high-strength fasteners assemblies shall conform to the applicable provisions of their ASTM standard. Besides, the Group C is limited to specific building locations and non-corrosive environmental conditions by the applicable ASTM standard.

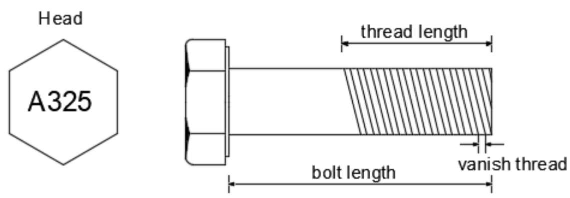


Figure 11: A325 high-strength structural bolt.

3.3.3 Snug-tight, Pretensioned and Slip-critical Bolts

- **Snug-tight bolts**. It is the situation existing when all the plies of a connection are in firm contact with each other. Snug-tight bolts are permitted for all the situations in which pretensioned or slip-critical bolts are not required. In this type of connection, the plies of steel being connected must be brought together so that they are solidly seated against each other, but they do not have to be in continuous contact [3].
  Bolts are permitted to be installed to the snug-tight condition when used in [3]:
  1. Bearing-type connections, except as stipulated in Section E6 in Specification for Structural Steel Buildings.
  2. Tension or combined shear and tension applications, for Group A bolts only, where loosening or fatigue due to vibration or load fluctuations are not design considerations.
- **Pretensioned bolts**. The bolts in a pretensioned joint are brought to very high tensile stress equal to approximately 70 percent of their minimum tensile stresses. The bolts must be pretensioned according to the following conditions [3]:

1. As required by the *RCSC Specification*.
2. Connections subjected to vibratory loads where bolt loosening is a consideration.
3. End connections of built-up members composed of two shapes either interconnected by bolts, or with at least one open side interconnected by perforated cover plates or lacing with tie plates, as required in Section E6.1 in Specification for Structural Steel Buildings.

- **Slip-critical bolts**. The installation of slip-critical bolts is identical with that for pretensioned joints, the difference lies on the treatment of the contact or faying surfaces. Slip-critical joints are needed in cases involving shear or combined shear and tension. The following connections shall be designed as slip-critical [3]:
1. As required by the *RCSC Specification*.
2. The extended portion of bolted, partial-length cover plates, as required in Section F13.3 in Specification for Structural Steel Buildings.

3.3.4 Proof Load and Bolt Tension for Slip-critical Connections

Until the 1985 RCSC Specification, all high-strength bolts were required to have a sufficient pretension force to create as high a compression force as practical between the pieces being connected, such that shear forces were transmitted through connections by friction between the connected pieces.
The proof load stress is approximately a minimum of 70% and 80% of the minimum tensile stress for A325 and A490 bolts, rounded off to the nearest kip as shown in the next table.

Bolt diameter (in)	A325 Bolts	A490 Bolts
1/2	12	15
5/8	19	24
3/4	28	35
7/8	39	49
1	51	64
1 1/8	56	80
1 1/4	71	102
1 3/8	85	121
1 1/2	103	148

3.4 Welding

3.4.1 Basic Processes for Welding

Welding is the process where metallic materials are jointed by heating their surfaces to a plastic or fluid state and allowing to flow the parts together until it becomes merged in one material. In process, either there may or may not be pressure, and for that, there may or may not be filler material employed [1].
In structural steel, it is used in an extensive way, SAW and SMAW despite there are other procedures such as the following:

- Gas Metal Arc Welding (GMAW).
- Fluxed Cored Arc Welding (FCAW).
- Electrogas Welding (EGW).
- Electroslag Welding (ESW).

All last processes have specified and particular standards in their implementation. Besides, they require qualified workforce to guarantee such standards related to quality that will be reflected in reliability and safety of the process carried out.

Shielded Metal Arc Welding (SMAW)

SMAW is one of the oldest methods for welding. It is even known as the simplest and most versatile for joining. A coated electrode is consumed as the metal is deposited from the electrode to the base material during welding process. The function of coating is to protect the electrode until the moment of melting. The heat produced by the arc induces that the end of the electrode burns and melts to make up an environment to produce the transference of metal given by the electrode wire and base metal. This process is carried on among a shielding gas.
The coating is clay-like mixture of silicates binders and powdered materials, such as fluorides, carbonates, oxides, metal alloys, and cellulose. This mixture is extruded and heated to produce a dry, hard, concentric coating.

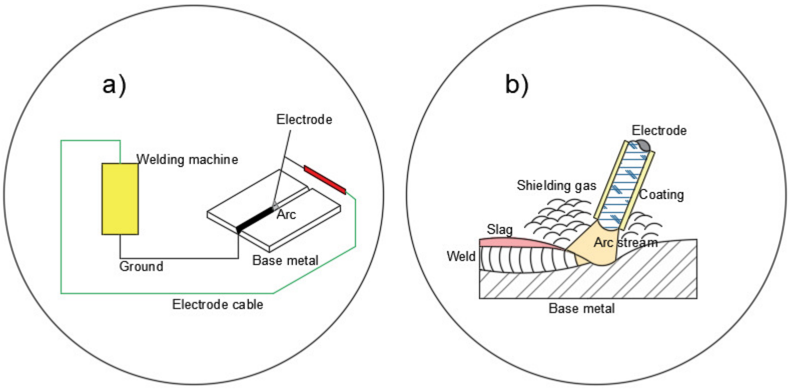


Figure 12: SMAW procedure: a)Arc welding circuit and b)Shielded arc welding.

Submerged Arc Welding (SAW)

This weld has the particular feature for application of certain granular material denominated *flux*. The end of the electrode is completely shielded by the molten flux over which is deposited a layer of unfused flux in its granular shape. The flux provides a cover that allows the weld to be made without spatter, sparks, or smoke. Besides, the use of flux material protects the weld pool against atmosphere, serves to clean the weld metal, and modifies the chemical composition of the weld metal.
The flux is a mixture of several minerals with determined features of slagging and viscosity. Those welds made by use of submerged arc welding have uniformity and high quality; exhibiting good ductility, high impact strength, high density and good corrosion resistance.

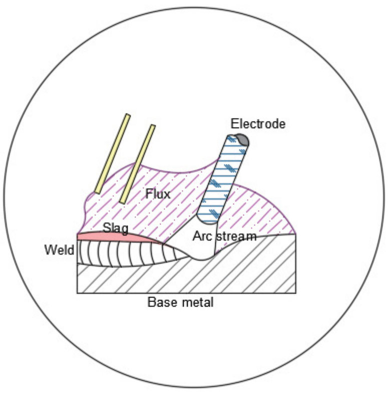


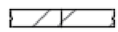
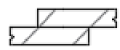
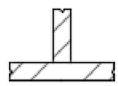
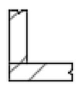
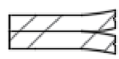
Figure 13: SAW procedure.

3.4.2 Advantages and of Weld

The advantages of welds respect to fastener elements are diverse. The following are just some of the most practical [1]:

- The use of welds reduces fastener elements, such as union plates, bolts, overlap plates, and other elements which are not necessary to be fixed.
- Connections for tension members are really functional for weight saving.
- Welding is the only process whose unions are totally hermetic.
- The welding process is much less noisy compared to assembly of bolted connections.

3.4.3 Types of Joints

Types of joints	
Butt. The butt joint is mainly used to join plates of the same or nearly the same thickness. It is mostly used to eliminate eccentricities developed in a single lap joints. The plates to be connected must be correctly aligned.	
Lap. The edges of the pieces that are overlapped do not require any special handling. A further advantage is the ease in which plates of different thickness can be joined. Besides, it shares the ease of fitting and ease of joining. It is also useful for hiding manufacturing errors.	
Tee. It is mainly seen in building of built-up sections, Tees, I-shapes, bearing stiffeners, hangers, among others.	
Corner. These are very common for rectangular box sections such as used for columns and beams that will be subjected to considerable torsional forces.	
Edge. In general, edge joints are not structural but are mostly used to keep two or more plates in a given plane.	

3.4.4 Types of Welds

There are four types of welds: the groove, fillet, slot and plug welds (Figure 14). Each of them owns properties and certain advantages that determine the extent of its use.

Groove Welds

They make up about 15% of structural weld. Due to costs of preparation, groove welds are more expensive than fillet welds. In fact, groove welds cost up to 50 to 100 percent more than fillet

welds [3]. There are two variations of groove welds: *complete-penetration* and *partial-penetration*. The partial-penetration welds only extend to a part of the whole thickness of member that will be connected.
The principal use of groove welds is to connect structural members that are aligned in the same plane. Another important fact is that groove welds represent an economic solution for strength to heavy loads.

Fillet Welds

Fillet welds join two surfaces frequently in right angle. In theory, they have a triangular cross-section, and they are very recurrent in tee, corner, and lap joints. This configuration is proposed for light loads because of less requirements for its preparation and process being at the same time an economical solution. Fillet welds are used in extensive way, with a figure around 80% of structural weld [1].

Slot and Plug Welds

The gaps may be filled with partial or completely depending on the plate thickness. This configuration is not consider as a primary welds due to its particular use to gain additional strength in situations where there is no space left to apply primary welds.
Slot and plug welds have the principal function of transferring shear in a lap joint when the size limits the length of weld necessary. Also, another important function is avoiding overlapping parts from buckling [1].

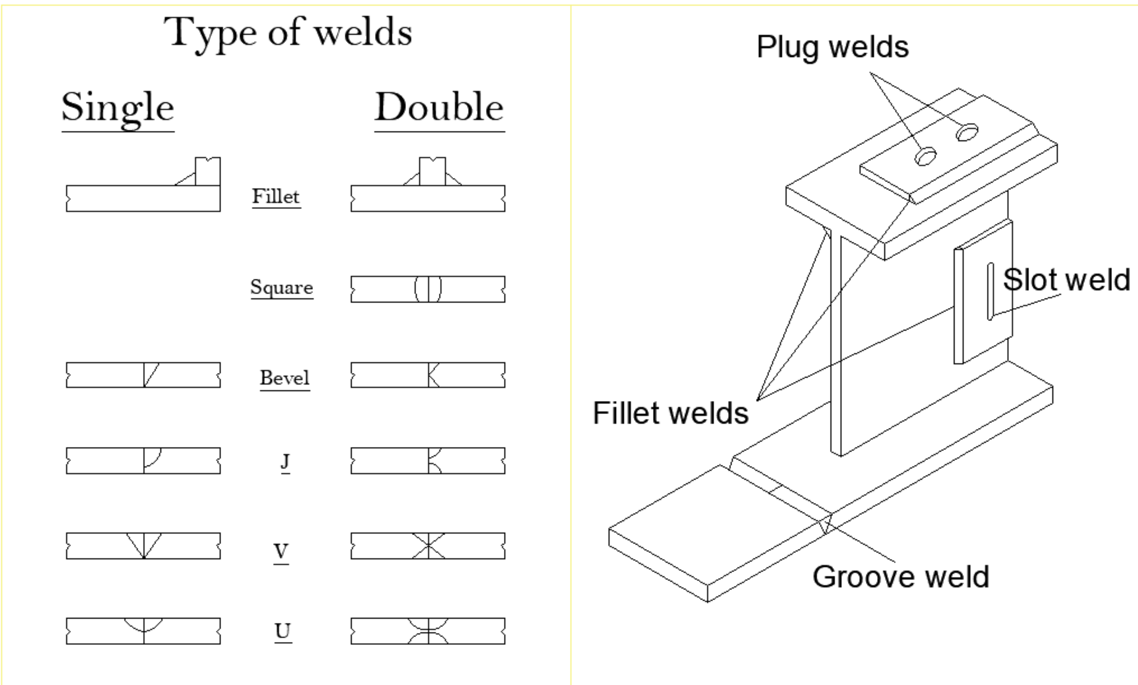


Figure 14: Weld variations.

3.4.5 Minimum Size of Fillet Welds

There is a minimum fillet weld size given by the AISC Specification. This is a first measure when it is considered all the effects generated by heating the weld and after the cooling. It may appear some cracks over the weld due to contraction when the thicker material is being welded, besides,

if the weld is considered short, then the hole quantity of heating could disappear when welding process is carried on along the edges, and there is going to be a certainty if both members to join are very thick [1]. This cooling have repercussions in a loss of ductility.
The minimum permissible size of fillet welds are given by AISC Specification (Table J2.4) in the next table.

Minimum Size of Welds w_{min}	
Material thickness of the thinner part joined, in (mm)	Minimum size of fillet weld, in (mm)
To 1/4 (6) inclusive	1/8 (3)
Over 1/4 (6) to 1/2 (13)	3/16 (5)
Over 1/2 (13) to 3/4 (19)	1/4 (6)
Over 3/4 (19)	5/16 (8)

3.4.6 Maximum Size of Fillet Welds

Sometimes, the features and dimensions of union elements do not allow filling suitably along elements. Besides, when the weld inspector carries such inspection on, it will be necessary to identify the edge [1]. Hence, it is important to take requirements and providing to finish the throat thickness of weld at least $1/16in$ before corner. Such dimension must accomplish that is within the range of allowable values.
According to last:

$$w_{max} = \begin{cases} (t_p) \ \ if \ \ t_p < \dfrac{1}{4}in \\ (t_p - \dfrac{1}{16}in) \ \ if \ \ t_p \geq \dfrac{1}{4}in \end{cases}$$

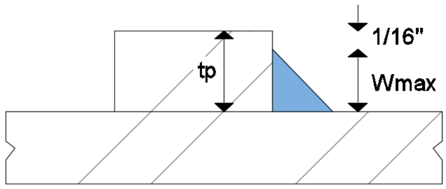


Figure 15: Maximum fillet weld size.

3.4.7 Limitations of Fillet Welds

Fillet welds are subjected to the next requirements:

- The minimum size of welds will not be less than the size required to support calculated forces, nor the sizes shown in table J2.4 from AISC Specification. These provision do not apply to fillet welds reinforcements of partial-joint-penetration (PJP) or complete-joint-penetration (CJP) groove welds.
- The minimum length of fillet welds designed must be upper than four times the nominal weld size ($4w$).
- The effective length of fillet welds will be computed as follows:
  - For end-loaded fillet welds with a length up to 100 times the weld size, it is permitted to take the effective length as the actual length.
  - When the effective length of the end-loaded fillet weld exceeds 100 times the weld size, the effective length will be computed by product of the actual length by some reduction factor, β, which is determined as:

$$\beta = 1.20 - 0.002\frac{l}{w} \leq 1.00 \tag{32}$$

Where l is the actual length of end-loaded fillet weld, and w is the size of weld leg.

  - If the length of fillet weld exceeds 300 times the size of weld leg, then, the effective length shall be taken as $180w$.

- Intermittent fillet welds are permitted to be used to transfer calculated stress across a joint or faying surfaces and join to components of built-up members.
  The length of any intermittent fillet weld will not be less than four times the size of weld leg, with a minimum of 1 $1/2in$ ($38mm$).

- In lap joints, the minimum amount of lap shall be five times the thickness of the thinner part joined, but not less than $1in$ ($25mm$). Lap joints joining plates or bars subjected to axial stress that utilize transverse fillet welds only shall be fillet welded along the end of both lapped parts, except where the deflection of the lapped parts is sufficiently restrained to prevent opening of the joint under maximum loading.

- Fillet welds in holes or slots are permitted to be used to transmit shear and resist loads perpendicular to the faying surface in lap joints or to prevent the buckling or separation of lapped parts and to join components of built-up members. Such fillet welds are permitted to overlap, subject to the provisions of Section J2. Fillet welds in holes or slots are not to be considered plug or slot welds.

- For fillet welds in slots, the ends of the slot shall be semicircular or shall have the corners rounded to a radius of not less than the thickness of the part containing it, except those ends which extend to the edge of the part.

3.4.8 Minimum Size of Groove Welds

For groove welds, according to AISC Specification (Table J2.3) [10]:

Minimum Size of Partial Penetration Welds	
Material thickness of the thinner part joined, in (mm)	Minimum size of fillet weld, in (mm)
To 1/4 (6) inclusive	1/8 (3)
Over 1/4 (6) to 1/2 (13)	3/16 (5)
Over 1/2 (13) to 3/4 (19)	1/4 (6)
Over 3/4 (19) to 1 1/2 (38)	5/16 (8)
Over 1 1/2 (38) to 2 1/4 (57)	3/8 (10)
Over 2 1/4 (57) to 6 (150)	1/2 (13)
Over 6 (150)	5/8 (16)

3.4.9 Electrodes and Classification

By heating the electrode or filler material, this will be melted with the base material also in a heated state, and both in a liquid form will be later allowed to cool. In that way, the filler material will control the final composition and mechanical properties of weld [1].
This electrode has a specific and exclusive chemical composition to reach equalizing the base material properties after solidification due to cooling. According to last, the base metal will dictate a strength parameter where the electrode to employ will have at least the same, even though the electrodes to use may have superior strengths, it also affects in a ductility reduction. Below there is a table shown, adapted from table 4.1.1 of American Welding Society (AWS 2000) which matches the most likely electrode for a determined base material located in each group.

Group where base material is founded	**SMAW process**	**SAW process**
I	E60XX or E70XX	F6X or F7X
II	E70XX	F7X
III	E80XX	F8X
IV (thickness greater than 2 $1/2in$)	E100XX	F10X
V (thickness less or equal to 2 $1/2in$)	E110XX	F11X

Where:

- E is for electrode.
- F refers to flux.
- The numbers located after E or F denote the minimum tensile stress of the filler material (ksi).
- XX for E, where the first X will be defined for certain number which expresses the use of electrode as the below is shown:
  - 1.- indicates that electrode may be used in any position.
  - 2.- expresses that electrode can only be used in flat and horizontal weld.
  - 3.- means that electrode will be exclusively used in flat position.

  Otherwise, the second X refers to the type covering, the type of current for process and polarity.

3.4.10 Effective Area for Welds

Fillet welds

For fillet welds, either they were made in a continuous or intermittent line, the effective area will be calculated as follows [1]:

$$A_{we} = L_{we} \cdot t_e \tag{33}$$

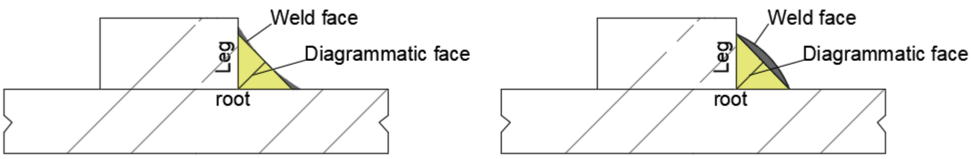


Figure 16: Concave and convex surface in fillet welds.

Where:

- L_{we} = effective length of fillet weld. This figure is computed according to length of weld over the size of weld leg parameter (see Limitations of Fillet Welds).
- t_e = *effective throat thickness* of the weld. The effective throat thickness of a fillet weld is defined as the shortest distance from the root to the theoretical (also known as diagrammatic) face of weld.

For fillet welds made by SMAW process and assuming that the leg size is the same, the effective throat thickness shall be computed as below [1]:

$$t_e = 0.707w$$

Otherwise, those fillet welds made by SAW process, the effective throat thickness will be:

- If the leg size is equal or less than $3/8in$ ($9.5mm$), the throat thickness shall be taken as equal to the actual leg size w.
- For a leg size equal or greater than $3/8in$ ($9.5mm$), the throat thickness will be the quantity $0.707w$ plus the amount of $0.11in$.

Groove welds

The effective area of groove welds shall be taken as the length of the weld times the effective throat [10]. Therefore, the expression would be the same that Equation 33, but in this case without computing effective length, and the effective throat thickness shall be considered regarding to the configuration of groove weld used and welding procedure.
The effective throat of a CJP groove weld shall be the thickness of the thinner part joined.
There is also important to notice that the considerations for effective throat thickness of PJP. Such previsions are shown below [2]:

- When bevel- or V-joint grooves have an included angle within 45 and 60, for SAW procedure in any position, or when the GMAW or FCAW processes are used in vertical or overhead positions, the effective throat is the depth of chamfer less the quantity of $1/8in$.
- Either the welding process is GMAW or FCAW, the effective throat for 45 bevel in flat or horizontal position will be the depth of groove.
- For welds made in flat position by SAW process, and the type of groove weld is a J, U, 60 bevel or V, the effective throat will also be the depth of groove as the last sentence.
- For SMAW, GMAW or FCAW process in any position, and having a J, U or 60V configuration of groove weld, the effective throat thickness shall be taken as the depth of groove.

Such considerations are included in AISC Specification (Table J2.1).

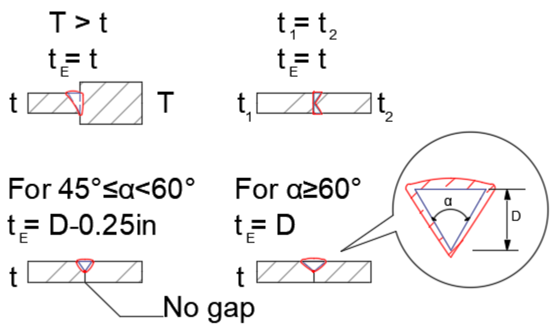


Figure 17: Effective throat dimensions for partial joint penetration groove welds (PJP).

Table J2.2 Effective throat of flare groove welds.

Welding Process	Flare Bevel Groove	Flare V-Groove
GMAW, FCAW-G	$\frac{5R}{8}$	$\frac{3R}{4}$
SMAW, FCAW-S	$\frac{5R}{16}$	$\frac{5R}{8}$
SAW	$\frac{5R}{16}$	$\frac{R}{2}$
[a] For $R < 3/8in(10mm)$, use only reinforcement with fillet welds. Note: R is equal to the joint surface (permitted $2t$ for HSS).		

The effective throat of a CJP groove weld shall be the thickness of the thinner part joined [10].

3.4.11 Nominal Strength for Welds

According to AISC-360-16 Specifications, the strength of welds are determined as below:

a) The strength design, and the allowable strength of welded joints shall be the lower value of the base material strength determined according to the limit states of tensile rupture and shear rupture and the weld metal strength determined according to the limit state of rupture as follows:

For the base metal

$$R_n = F_{nBM} \cdot A_{BM} \tag{34}$$

For the weld metal

$$R_n = F_{nw} \cdot A_{we} \tag{35}$$

Where:

- A_{BM} = cross-sectional area of the base metal.
- A_{we} = effective area of the weld.
- F_{nBM} = nominal stress of the base metal.
- F_{nw} = nominal stress of the weld metal.

The values of ϕ, Ω, F_{nBM} and F_{nw}, and corresponding limitations are given in Table J2.5.

b) For fillet welds, the available strength is permitted to be determined accounting for a directional strength increase of $(1 + 0.50sin^{1.5}\theta)$ if strain compatibility of the various weld elements is considered.

Where:

- $\phi = 0.75$ (LRFD), $\Omega = 2.00$ (ASD).
- θ = angle between line of action of the required force and the weld longitudinal axis.

(1) For a linear weld group with a uniform leg size, loaded through the center of gravity will be computed as equation 35.

Where:

- $F_{nw} = 0.60F_{EXX}(1 + 0.50sin^{1.5}\theta)$
- F_{EXX} = filler metal strength.

(2) For fillet weld groups concentrically loaded and consisting of elements with a uniform leg size that are oriented both longitudinally and transversely to the direction of applied load, the combined strength, R_n, of the fillet weld group shall be determined as the greater value of the following expressions:

$$R_n = R_{nwl} + R_{nwt} \tag{36}$$

or

$$R_n = 0.85R_{nwl} + 1.50R_{nwt} \tag{37}$$

Where:

- R_{nwl} = total nominal strength of longitudinally loaded fillet welds, as determined in accordance with Table J2.5.
- R_{nwt} = total nominal strength of transversely loaded fillet welds, as determined in accordance with Table J2.5 without the increase of $(1 + 0.50sin^{1.5}\theta)$.

3.4.12 Standard Welding Symbols

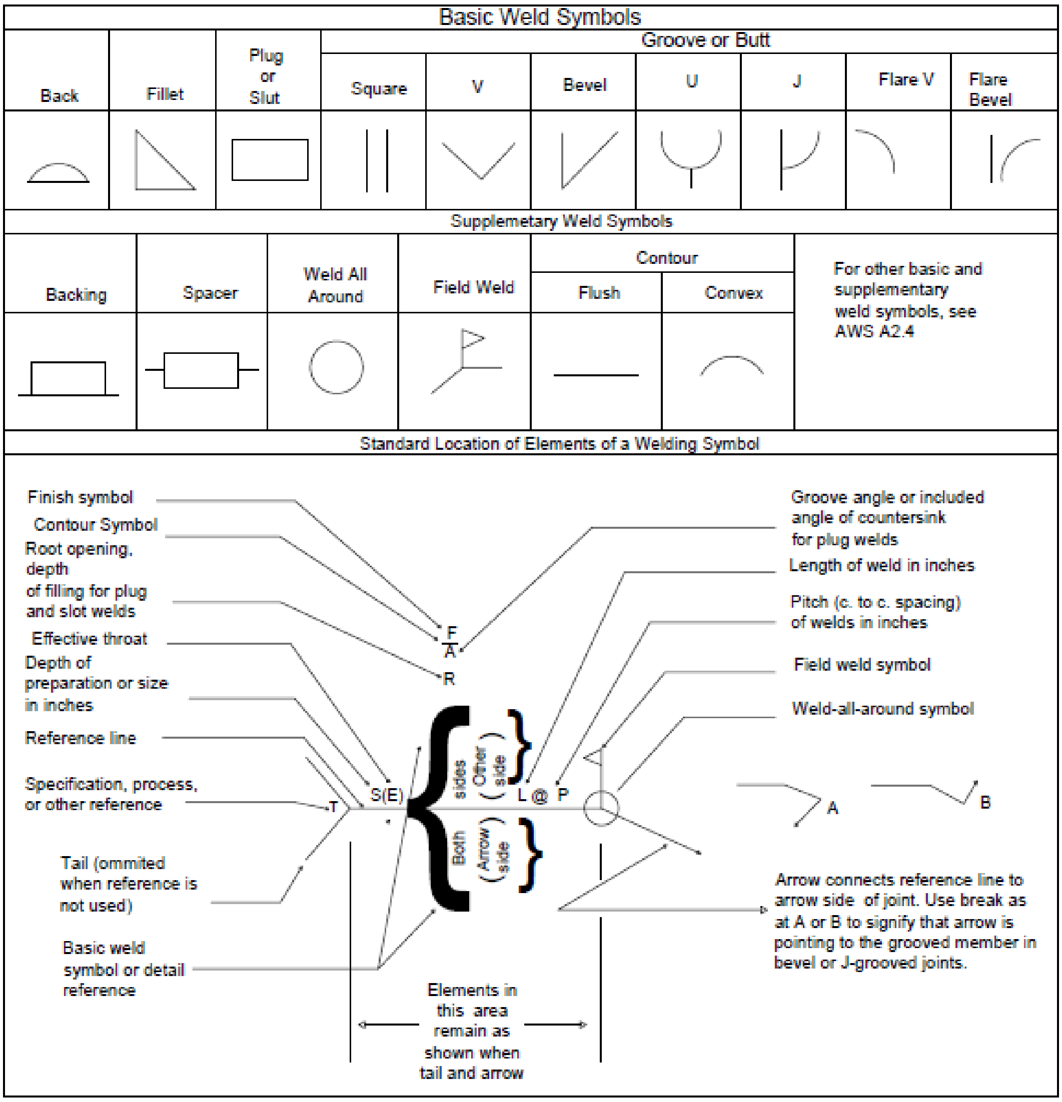


Figure 18: Standard welding symbols.

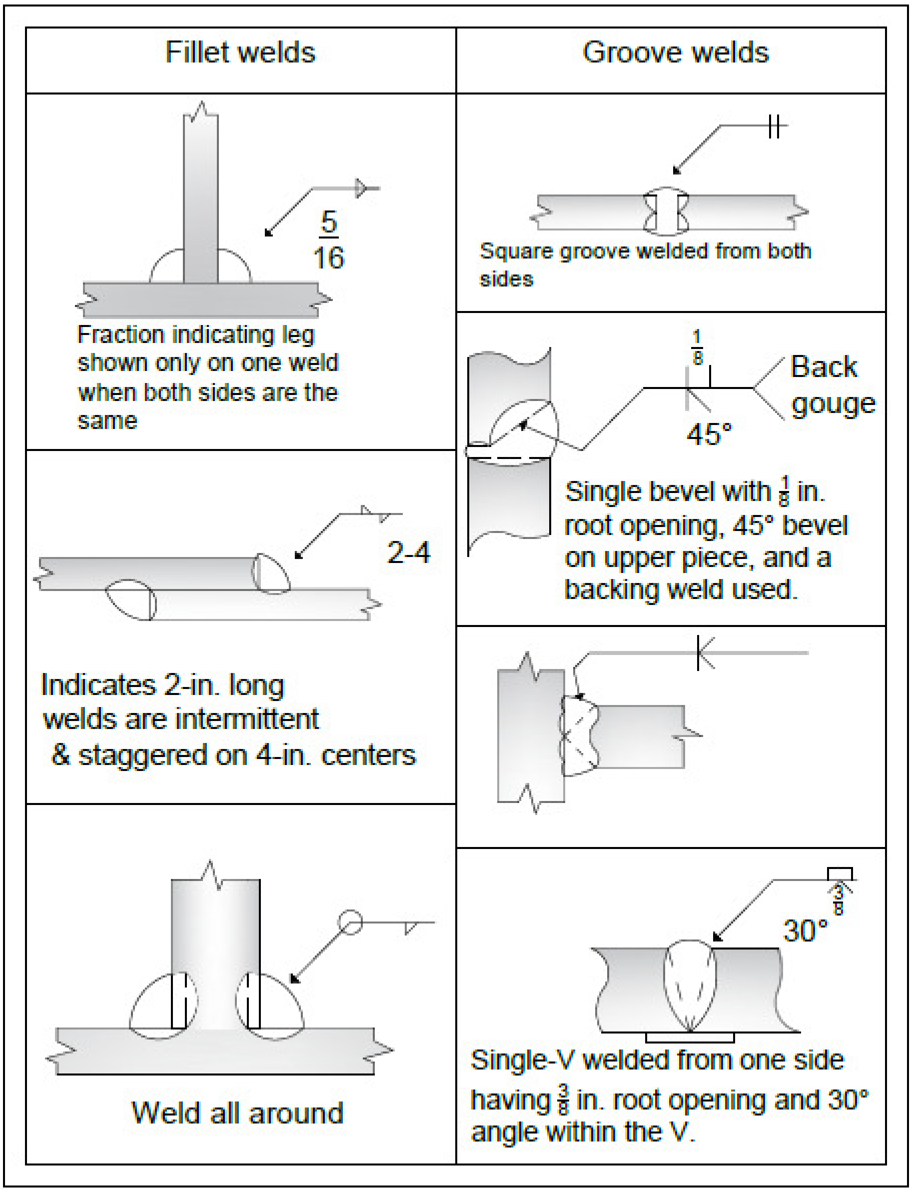


Figure 19: Common uses of welding symbols.

3.5 Rigid Steel Connections

3.5.1 Introduction

Theoretically, there are not perfect rigid or simple ended connections. Both of them behave as partially restrained. Fully restrained (FR) connections are idealized as rigid enough to keep almost the original angle after the loads being applied [1]. Such connections provide continuity between frames and all building in general.
For beam-column connections with W shapes, shear is taken by the web and bending moment by the flanges of its cross-section. Under static loads, and as long as these have the direction of gravity force, the bending moment is expected to be negative at fixed ends. Bending moment might be decomposed into a couple of forces (Figure 20) acting on the flanges, which is given by the next expression:

$$P_{uf} = \frac{M_u}{h} \tag{38}$$

Where:

- M_u = bending moment around major-axis of cross-section under factorized loads.
- h = depth of beam.
- P_{uf} = factorized force on the flanges of the W shape.

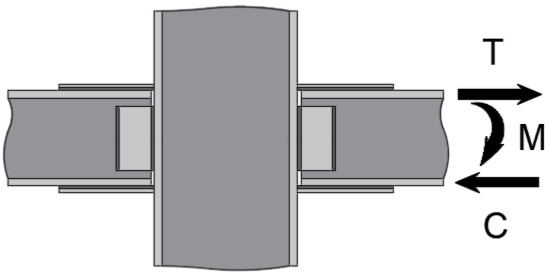


Figure 20: Beam-column connection with moment-resisting plates and shear plates joined by welds.

Such couple of tension-compression forces correspond to the value of P_{uf}. Under all mentioned considerations before, in practice, the upper plate is designed as a tension member and the lower as a member subjected to compression. The shear that would be resisted by the web, it is taken now by the shear plates that are welded to the flanges of the column.
The previous connection is one of many existing configurations, and the unions might be either welded or bolted. Even so, it is one of the most used pre-qualified configurations.

3.5.2 Limit States in FR Connections with Moment-resisting Plates

For this particular research job, we shall be focused on steel connections with welded unions, so, the principal limit states to analyze are the following [1]:

- For upper plate:
  - Yielding.
  - Fracture.
  - Strength of fillet welds.
  - Strength of groove weld.
- For shear plates:
  - Yielding.

  - Fracture.
  - Strength of fillet welds.
- For lower plate:
  - Strength of member considered as a column.

3.5.3 Yield Moment M_y and Plastic Moment M_p

To get the relation between stresses due to bending moment and corresponding strains for elements type beam, it is necessary being aware of the next assumptions [1]:

- Bending moment is located on plane of symmetry.
- There are not axial forces along the element.
- The cross-sections before loading are originally flat and perpendicular to longitudinal axis, then, after bending they also remain flat and perpendicular.
- The stress-strain diagram is idealized for two straight lines (Figure 21, b). Before reaching the yield stress, the material behaves linearly elastic. Beyond yield stress, strains are going to increase with no longer increase in stress. According to last, even so the process of loading wanted to be reverted, there will be a permanent deformation.
- The beam material owns the same properties for tension and compression.

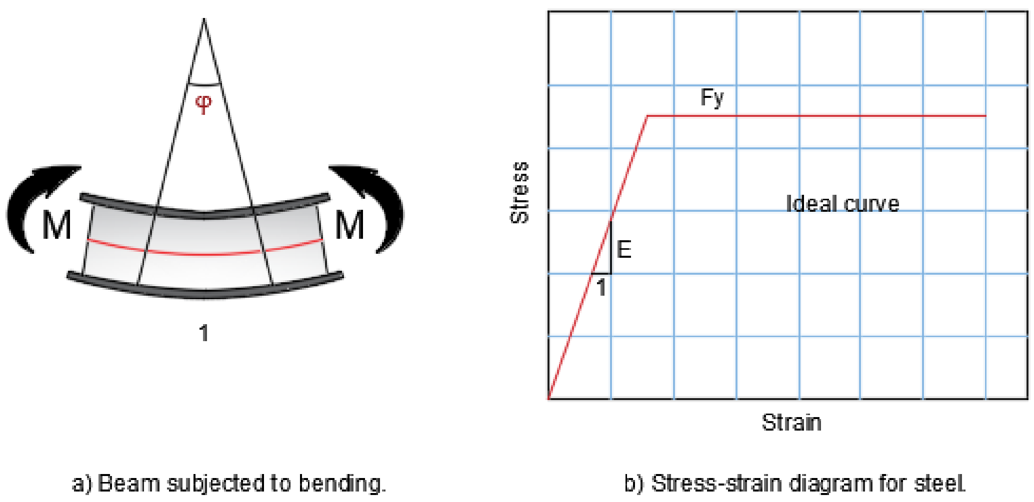


Figure 21: Beam subjected to bending with unitary length.

The beam elements considered for last assumptions have rectangular cross-sections, W shapes, and one single symmetric axis.
The rotation will be measured by a ϕ parameter. It denotes the change of the angle between its original shape (unloaded) and deformed position after increasing of bending moment in end sections of unitary length. As long as the member behaves elastic, and according to mechanic of materials (Timoshenko and Young 1962):

$$\phi = \frac{M}{EI} = \frac{\varepsilon}{y}$$

W Sections

The stress distribution on a typical wide-flange shape considering residual stresses is shown in Figure 22 and consequently related to digram shown by Figure 23. Notice that for a moment under service loads is located in elastic zone, a clear example could be defined by stage **a**.
When the yield stress is reached (stage **b**) by the extreme fiber of cross-section, then, the nominal moment strength M_n becomes the *yield moment* M_y which is computed as

$$M_n = M_y = S_x \cdot F_y \tag{39}$$

Where:

- F_y = specified minimum yield stress.
- S_x = modulus section of the cross-section.

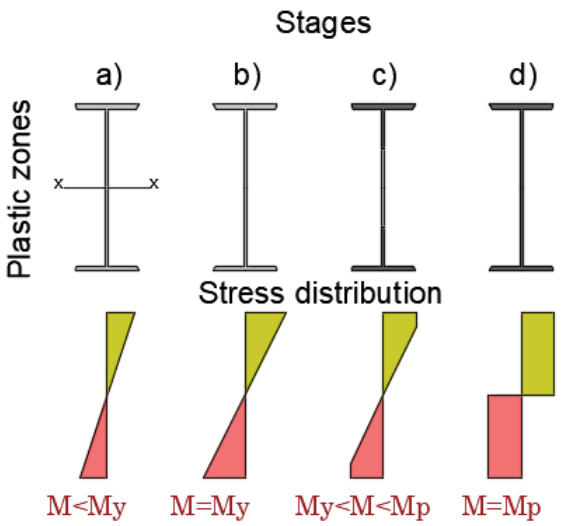


Figure 22: Stress distribution in wide-flange shape after increasing bending moment.

The stage **d** is reached when every fiber in the cross-section has a strain equal to or greater than ε_y. Hence, it is known that the element is in the plastic range. Therefore, the nominal moment is now computed by

$$M_p = F_y \int_A y \, dA = Z_x \cdot F_y \tag{40}$$

Where:

- Z_x = *plastic modulus* of the cross-section.

The ratio known as *shape factor* (ψ) is a property independent of the element material. This ratio is given by [2]:

$$\psi = \frac{M_p}{M_y} = \frac{Z_x}{S_x} \tag{41}$$

In case for W sections, the shape factor ranges from 1.09 to about 1.18, with a usual values around of 1.12. According to last, it could be summarized conservatively that plastic moment strength (the cross-section behaves entirely plastic) is at least 10% greater than the yield moment [2].
Once the plastic moment strength is reached, the section can not offer additional resistance to rotation. Then, it recreates a condition of *plastic hinge* with a constant resistance M_p.
It is very important realizing that the real stress distribution over the cross-section is totally nonlinear, and the uniform distribution previously studied as a block is just an assumption for the approaching of plastic moment and ease for subsequent calculations.

In real beams, the residual stresses contribute to reaching yielding soon, and therefore, reducing the yielding moment [1]. However, the residual stresses do not reduce the plastic moment of the section. This effect might be seen in the green curve in Figure 23, loading gradually a W section from elastic to the plastic regime. The red curve represent the theoretic perfect elastoplastic behavior of the beam which would be expected without the effect of residual stresses.

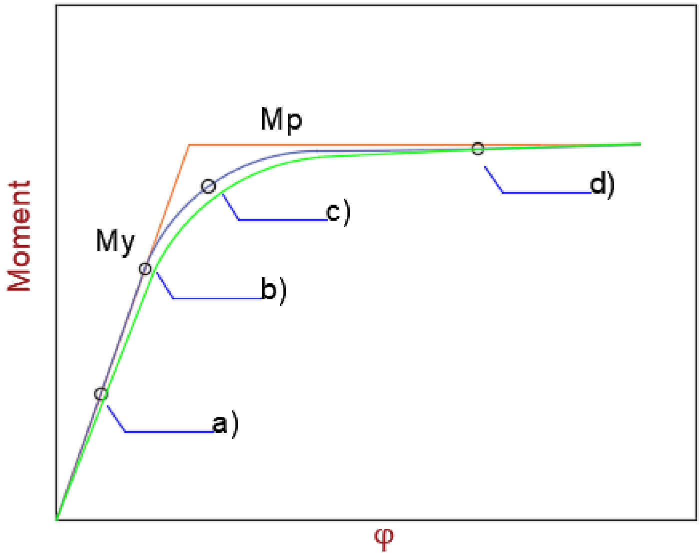


Figure 23: Stages of a W shape gradually taken to the plastic regime.

The respective strength reduction factor for flexural strength are given in the next way

- $\phi = 0.90$ (LRFD), $\Omega = 1.67$ (ASD).

4 Finite Elements Method

4.1 Theoretical Basis

4.1.1 Strain Measures

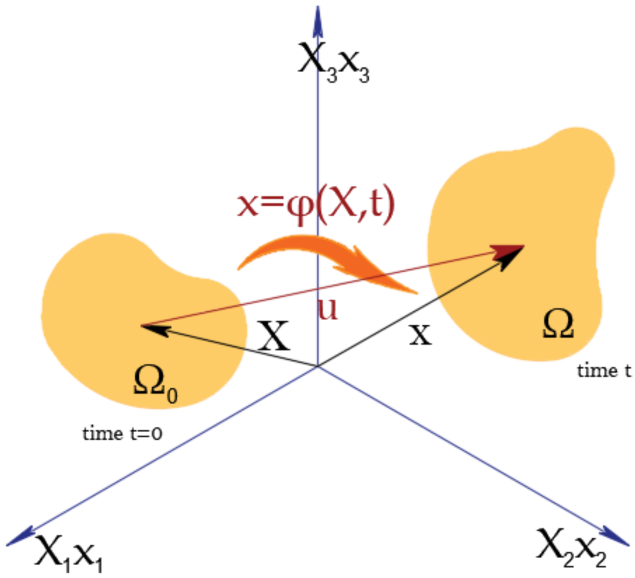


Figure 24: Motion and strain of continuum body $\mathcal{B}$.

In field of study about solids mechanics, a continuum body $\mathcal{B}$ will experiment a deformation in its geometry due to forces acting on it. Strain measures such deformations representing the rate of initial configuration Ω_0, and current configuration Ω of the body. Therefore, the strain expresses itself motion and deformation.
There are many kinematic measures of strain in continuum mechanics. For Lagrangian descriptions which are employed in study of solids mechanics, the strain is described by *Green-Lagrange strain tensor* shown as below:

$$E = \frac{1}{2}\left(F^T \cdot F - I\right) \quad or \quad E_{ij} = \frac{1}{2}\left(F^T_{ik} \cdot F_{kj} - \delta_{ij}\right) \tag{42}$$

Where:

- E is the Green-Lagrange strain tensor.
- F is a primary measure in nonlinear mechanics known as *material gradient tensor* given by

$$F = \frac{\partial x}{\partial X} \quad or \quad F_{ij} = \frac{\partial \phi_i}{\partial X_j} = \frac{\partial x_i}{\partial X_j} \tag{43}$$

  which is associated to the rate of material or initial configuration and spatial or current configuration. The material gradient tensor **F** is also known as *Jacobian matrix* [6].
- I otherwise expressed as δ_{ij} *(Kronecker delta)* wich values are

$$\delta_{ij} = \begin{cases} 1 & if \quad i = j \\ 0 & otherwise \end{cases}$$

Equation 42 also may be expressed in function of displacement gradient tensor by

$$E_{ij} = \frac{1}{2}\left(\frac{\partial u_i}{\partial X_j} + \frac{\partial u_j}{\partial X_i} + \frac{\partial u_k}{\partial X_i} \cdot \frac{\partial u_k}{\partial X_j}\right) \tag{44}$$

For study of solid problems based on infinitesimal strains ($\|J\| << 1$), the nonlineal term of last equation is neglected. In that way, there is left the *infinitesimal strain tensor*

$$\varepsilon_{ij} = \frac{1}{2}\left(\frac{\partial u_i}{\partial X_j} + \frac{\partial u_j}{\partial X_i}\right) \tag{45}$$

4.1.2 Stress

After motion and deformation of a continuum body $\mathcal{B}$, some forces are generated by interaction among its particles [6]. If a general deformable body as shown in Figure 24 is cut by any plane, the reactions will be represented as shown in Figure 25 so that effects on region R_1 must be consistent on region R_2.

Consider an element $d\Gamma$ normal to a unit vector **n** in the neighborhood of spatial point **P**. If the

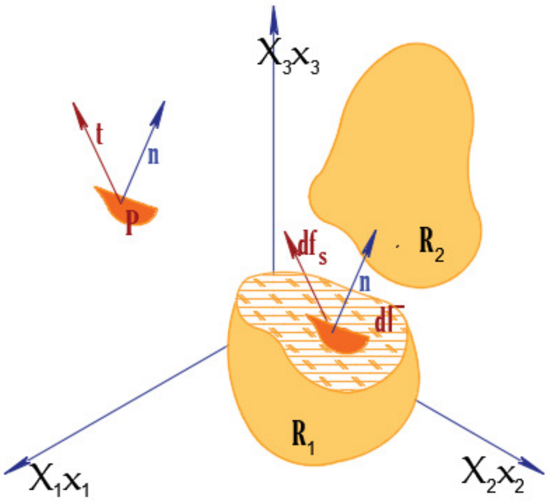


Figure 25: Traction vector.

resultant force on this area $d\Gamma$ is equal to df_s, the traction vector **t** corresponding to normal **n** at **P** is defined as

$$t(n) = \lim_{d\Gamma \to 0} \frac{df_s}{d\Gamma} \tag{46}$$

where the relationship between **t** and **n** satisfies third Newton's law of action and reaction, which it establishes that [7]

$$t(-n) = -t(n) \tag{47}$$

There is a spatial tensor field σ known as *Cauchy stress tensor* such that for every normal unit vector **n** exists

$$t = n \cdot \sigma = \sigma^T \cdot n \;\; or \;\; t_i = \sigma_{ji} \cdot n_j \tag{48}$$

4.1.3 Principle of Virtual Work

One of the fundamental variational principles is the *Principle of Virtual Work*. The principle of virtual work assumes arbitrary virtual displacements δu_i which are independent of time. Those virtual displacements have a null value at Dirichlet's boundary ($\delta u_i = 0 \; at \; \Gamma^D$) [6].
The external virtual work is given by the product of an arbitrary virtual displacement δu_i and the external forces over a body $\mathcal{B}$, and it may be expressed by

$$\delta W^{ext} = \int_\Omega \rho b \cdot \delta u \; d\Omega + \int_{\Gamma^N} t \cdot \delta u \; d\Gamma \tag{49}$$

where the first integral belongs to mass forces acting upon domain and the second for surface forces over the boundary.

Otherwise, internal virtual work made by the particles of a deformable body, and according to infinitesimal strains may be described by

$$\delta W^{int} = \int_{\Omega} \delta\varepsilon \cdot \sigma \; d\Omega \tag{50}$$

According with the last, for study of static problems, the equilibrium of the system can be explained by the next description

$$\int_{\Omega} \delta\varepsilon \cdot \sigma \; d\Omega = \int_{\Omega} \rho b \cdot \delta u \; d\Omega + \int_{\Gamma^N} t \cdot \delta u \; d\Gamma \tag{51}$$

The last equation is better known as *Weak Form* or *Variational Form* of internal equilibrium, and it requires of boundary conditions to obtain the solution. The difference respect to *Strong Form* is that strong form evaluates the solution over all entire domain, and eventually that leads to an exact solution. For that, to build and compute the solution of the equation, the approaching by weak form and employment of numerical methods can only give us an exact solution in some points over domain and the rest of them may be obtained by interpolation. According to last, it is summarized the particular features between an analytical and *discrete solution* [6].

4.1.4 Elasticity

The structural materials own an “elastic” property. This property displays when the strains disappear after removing external forces applied on some rigid body, as long as a certain limit is not exceeded. Another concepts that we might consider are isotropy and homogeneity. Homogeneity is accomplished when the features and properties of determined material are the same for every particle that composes it, and finally, isotropy refers to how vectorial magnitudes are equally distributed in all directions.
The essential hypothesis for elasticity are enlisted below [4]:

- **Small displacements**
  There is not a meaningful response between material and spatial configuration descriptions after the loads applied in the domain.
  $$x \approx X$$
  According to last, we might summarize:
  $$F = \frac{\partial x}{\partial X} \approx 1 \Longrightarrow \|F\| \approx 1$$
- **Existence of neutral state**
  It is allowed the existence of a neutral state where strains and stresses are totally null. The last can be expressed as:
  $$\begin{cases} \varepsilon(x, t_0) = 0 \\ \sigma(x, t_0) = 0 \end{cases}$$
- **The deformation process is isothermal and adiabatic**
  Isothermal processes are those that keep a constant temperature $\theta(x, t)$ over time.
  $$\theta(x, t) \equiv \theta(x)$$
  Adiabatic processes are those that can not allow heat conduction.
  $$\rho r - \nabla \cdot q = 0$$
  $$\begin{cases} \forall \;\; x \\ \forall \;\; t \end{cases}$$

4.1.5 Voigt's Notation

Due to symmetry of tension and deformation tensors, it results "economic" rewriting the last expressions in vectorial terms. This notation is for simplifying the mathematical operations and computational cost. This procedure is known as Voigt's notation, and consists of building the vectors with the upper triangle from the matrix components of the tensors (considering that those over the diagonal are equal to those under) [4]:

$$\sigma = \begin{bmatrix} \sigma_{xx} & \sigma_{xy} & \sigma_{xz} \\ \sigma_{yx} & \sigma_{yy} & \sigma_{yz} \\ \sigma_{zx} & \sigma_{zy} & \sigma_{zz} \end{bmatrix} \Longrightarrow \{\sigma\} = \begin{Bmatrix} \sigma_{xx} \\ \sigma_{yy} \\ \sigma_{zz} \\ \sigma_{xy} \\ \sigma_{xz} \\ \sigma_{yz} \end{Bmatrix} = \begin{Bmatrix} \sigma_{x} \\ \sigma_{y} \\ \sigma_{z} \\ \tau_{xy} \\ \tau_{xz} \\ \tau_{yz} \end{Bmatrix} \tag{52}$$

In the same way, for infinitesimal deformations tensor:

$$\varepsilon = \begin{bmatrix} \varepsilon_{xx} & \varepsilon_{xy} & \varepsilon_{xz} \\ \varepsilon_{yx} & \varepsilon_{yy} & \varepsilon_{yz} \\ \varepsilon_{zx} & \varepsilon_{zy} & \varepsilon_{zz} \end{bmatrix} \Longrightarrow \{\varepsilon\} = \begin{Bmatrix} \varepsilon_{xx} \\ \varepsilon_{yy} \\ \varepsilon_{zz} \\ \varepsilon_{xy} + \varepsilon_{yx} \\ \varepsilon_{xz} + \varepsilon_{zx} \\ \varepsilon_{yz} + \varepsilon_{zy} \end{Bmatrix} = \begin{Bmatrix} \varepsilon_{xx} \\ \varepsilon_{yy} \\ \varepsilon_{zz} \\ 2\varepsilon_{xy} \\ 2\varepsilon_{xz} \\ 2\varepsilon_{yz} \end{Bmatrix} = \begin{Bmatrix} \varepsilon_{x} \\ \varepsilon_{y} \\ \varepsilon_{z} \\ \gamma_{xy} \\ \gamma_{xz} \\ \gamma_{yz} \end{Bmatrix} \tag{53}$$

where axial deformations correspond to

$$\varepsilon_x = \frac{\partial u}{\partial x} \tag{54}$$

$$\varepsilon_y = \frac{\partial v}{\partial y} \tag{55}$$

$$\varepsilon_z = \frac{\partial w}{\partial z} \tag{56}$$

and tangential

$$\gamma_{xy} = \frac{\partial u}{\partial y} + \frac{\partial v}{\partial x} \tag{57}$$

$$\gamma_{xz} = \frac{\partial u}{\partial z} + \frac{\partial w}{\partial x} \tag{58}$$

$$\gamma_{yz} = \frac{\partial v}{\partial z} + \frac{\partial w}{\partial y} \tag{59}$$

4.1.6 Compatibility Equations for Linear Elasticity

The compatibility equations are those conditions that must validate a second-order symmetric tensor so that it can be a deformation tensor, and therefore, there is a displacements field where it comes from. The compatibility conditions establish the relation between stress and strain, and according to the dimension of the problem, they have an array of elastic constants that have been determined experimentally [4].
Such relation is known as *Hooke's Law*, and it might be expressed in its matrix form as

$$\sigma = \mathbb{C}\varepsilon \tag{60}$$

where $\mathbb{C}$ corresponds to the constitutive matrix or elastic constants matrix.
This notation is simplified due to it does not consider initial stresses and strains, temperature changes, residual stresses and so on. Given the Maxwell-Betti theorem and the premises of linear elasticity, it follows that $\mathbb{C}$ is strictly symmetric and

1. For one dimensional problems

$$\mathbb{C} = E \tag{61}$$

2. For plane stress problems in two dimensions

$$\mathbb{C} = \frac{E}{1-\nu^2}\begin{bmatrix} 1 & \nu & 0 \\ \nu & 1 & 0 \\ 0 & 0 & \dfrac{1-\nu}{2} \end{bmatrix} \tag{62}$$

3. For plane strain in two dimensions

$$\mathbb{C} = \frac{E(1-\nu)}{(1+\nu)(1-2\nu)}\begin{bmatrix} 1 & \dfrac{\nu}{1-\nu} & 0 \\ \dfrac{\nu}{1-\nu} & 1 & 0 \\ 0 & 0 & \dfrac{1-2\nu}{2(1-\nu)} \end{bmatrix} \tag{63}$$

4. For three dimensional problems

$$\mathbb{C} = \frac{E(1-\nu)}{(1+\nu)(1-2\nu)}\begin{bmatrix} 1 & \dfrac{\nu}{1-\nu} & \dfrac{\nu}{1-\nu} & 0 & 0 & 0 \\ \dfrac{\nu}{1-\nu} & 1 & \dfrac{\nu}{1-\nu} & 0 & 0 & 0 \\ \dfrac{\nu}{1-\nu} & \dfrac{\nu}{1-\nu} & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & \dfrac{1-2\nu}{2(1-\nu)} & 0 & 0 \\ 0 & 0 & 0 & 0 & \dfrac{1-2\nu}{2(1-\nu)} & 0 \\ 0 & 0 & 0 & 0 & 0 & \dfrac{1-2\nu}{2(1-\nu)} \end{bmatrix} \tag{64}$$

4.1.7 Introduction to Nonlinear Analysis

Despite the obvious success of the assumption of linearity in engineering analysis, it is equally obvious that many situations demand consideration of nonlinear behavior. But in the linear case an assumption is made that the deformation is sufficiently small to enable the effect of changes in the geometrical configuration of the solid to be ignored, whereas in the nonlinear case the magnitude of the deformation is unrestricted [7].
Two sources of nonlinearity exist in the analysis of solid continua, namely, *material* and *geometric* nonlinearity. The former occurs when, for whatever reason, the stress strain behavior given by the constitutive relation is nonlinear, whereas the latter is important when changes in geometry, however large or small, have a significant effect on the load deformation behavior. Material nonlinearity can be considered to encompass contact friction, whereas geometric nonlinearity includes

deformation-dependent boundary conditions and loading [7].
The discussion about the basic assumptions of linear analysis will make a reference point to understand the nonlinearity concept and how to identify and categorize different nonlinear analyses. The following table gives clearly the classification of nonlinear analyses that considers separately the material and kinematic nonlinear effects (Table 6.1 from Finite Element Procedures, Klaus-Jürgen Bathe).

Type of analysis	**Description**	**Typical formulation used**	**Stress and strain measures**
Materially-nonlinear-only	Infinitesimal strains and displacements; the stress-strain relation is nonlinear	Material-nonlinear-only (MNO)	Engineering stress and strain
Large displacements and rotations, but small strains	Displacements and rotations of fibers are large, but fiber extensions and angle changes between fibers are small; the stress-strain relation may be either linear or nonlinear	Total Lagrangian (TL) Updated Lagrangian (UL)	Second Piola-Kirchhoff stress, Green Lagrange strain ——— Cauchy stress, Almansi strain
Large displacements, large rotations, and large strains	Fiber extensions and angle changes between fibers are large, fiber displacements and rotations may also be large; the stress-strain relation may be linear or nonlinear	Total Lagrangian (TL) Updated Lagrangian (UL)	Second Piola-Kirchhoff stress, Green Lagrange strain ——— Cauchy stress, Logarithmic strain

To serve the purposes of this research, the nonlinearity studied will be focused on materially-nonlinear-only analysis. The principal concerns of this investigation using the MNO analysis are the following:

- Tensional state around welded joints.
- Structural performance of the steel connection subjected to a determined percentage of the plastic moment, and consequently make a comparison and contrast between several sets of configurations.
- Observe and studying the stresses distributed over the connections to predict and validate the practical methodologies given to get nominal strengths for FR connections as mentioned in, according to the structural performance expected to guarantee safety and resistance.

4.1.8 Elastoplastic Phenomenological Behavior

Elastoplastic models are employed in continuum mechanics to represent the behavior of materials when these either exceed certain limits in stresses or strains. The models studied lately will consider that strains are infinitesimal [4].
The study of *Plasticity* introduces two considerations respect to linear elasticity:

- Nonlinearity: stresses are not proportional to strains.
- Plastic deformation: the deformation caused by loading process is not totally recoverable.

Let us consider a steel bar with length l and a cross-section A subjected to a load F in conditions of a typical tension test. The stress σ will be the result of the magnitude F divided by the value of A, and the corresponding strain ε is equal to the elongation δ divided by the original length of the bar. This test will be carried on several times in loading and unloading cycles with the purpose of obtaining the $\sigma - \varepsilon$ response (Figure 26) [4].
As long as the elastic limit (σ_y) is not exceeded (0-1 stage), and the loading process is reverted, the caused deformations in the bar are recoverable to the starting configuration. Otherwise, for tensions higher than σ_y, the behavior is not elastic and the deformation generated can not be reverted at all. Now, it is observed that a *plastic strain* ε^P appears as result of described procedure. However, during the unloading cycle 2-3 the behavior becomes approximately elastic. If the

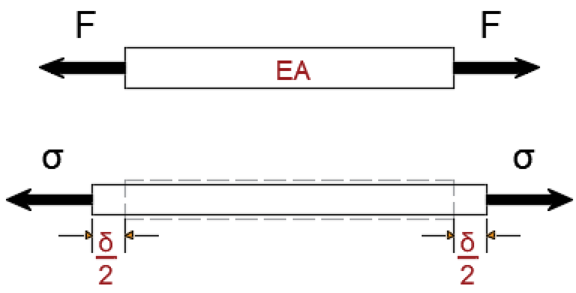


Figure 26: Load-unload-reload cycles

process of loading starts once again (3-2), now there is fixed a new yield point higher than the starting value σ_y (0-1). After repeating the loading and unloading cycles for 2-4-5-4, the response is similar to the previous cycle and there is more plastic strain accumulated during 2-4 stage [4].

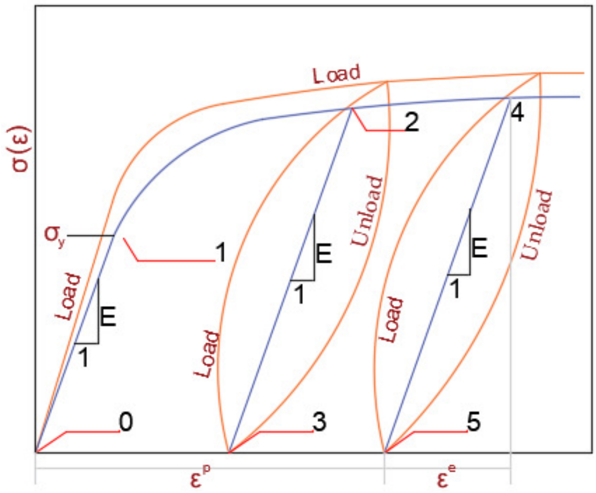


Figure 27: Load-unload-reload cycles

From the previous picture, diagram shows how the load-unload-reload cycle is gradually taken (orange stream) and the stress-strain being recorded (blue stream) as the response of the tension test. Further than yield stress, the curve becomes nonlinear and the total strain (for 0-1-2-4 before unloading) is the sum of plastic strain plus the elastic that is projected under the straight elastic line, therefore

$$\varepsilon = \varepsilon^p + \varepsilon^e \tag{65}$$

4.1.9 One-dimensional Incremental Theory of Plasticity

The elastoplastic phenomenological behavior previously seen, can be idealized and using mathematical models of certain complexity. One of the most popular approaches is the *incremental theory of plasticity*. For one dimension, it is pretended to approximate the stress-strain behavior by means of a piecewise function with an elastic and inelastic branches. For mayor dimensions, it is required the introduction of more abstract concepts [4].

Strain Decomposition and Hardening Variable

The total deformation ε it is splitted into the sum of elastic deformation ε^e (recoverable strain) which is ruled by Hooke's law, and the plastic deformation ε^p (remaining strain) [4].

$$\begin{cases} \varepsilon = \varepsilon^e + \varepsilon^p \\ \varepsilon^e = \dfrac{\sigma}{E} \end{cases} \Longrightarrow \begin{cases} d\varepsilon = d\varepsilon^e + d\varepsilon^p \\ d\varepsilon^e = \dfrac{d\sigma}{E} \end{cases}$$

Besides, it is defined the hardening variable $\alpha(\sigma, \varepsilon^p)$ by evolution equation [4]

$$\alpha \Longrightarrow \begin{cases} d\alpha = sign(\sigma) d\varepsilon^p \\ d\sigma \geq 0 \end{cases} \tag{66}$$

where the expression $sign(\sigma)$ corresponds to the sign function, and it denotes the values enlisted below according to the nature of the applied force (positive for tension and negative otherwise)

$$\begin{cases} sign(\sigma) = 1 \ if \ \sigma \geq \ 0 \\ sign(\sigma) = -1 \ if \ \sigma < 0 \end{cases}$$

Constitutive Equation

The material response will be located in one of the next situations [4]:

- Elastic regime:

$$\sigma \in \mathbb{E}_\sigma \Longrightarrow d\sigma = E \ d\varepsilon \tag{67}$$

- Elastoplastic regime due to unloading:

$$\left.\begin{array}{l} \sigma \in \partial\mathbb{E}_\sigma \\ dF(\sigma, \alpha) < 0 \end{array}\right\} \Longrightarrow d\sigma = E \ d\varepsilon \tag{68}$$

- Elastoplastic regime for plastic loading:

$$\left.\begin{array}{l} \sigma \in \partial\mathbb{E}_\sigma \\ dF(\sigma, \alpha) = 0 \end{array}\right\} \Longrightarrow d\sigma = E^{ep} \ d\varepsilon \tag{69}$$

  where E^{ep} is equal to the *elastoplastic deformation modulus.*

Hardening Parameter and Elastoplastic Deformation Modulus

The hardening law provides the evolution of plastic yielding stress $\sigma_f(\alpha)$ according to hardening parameter α. Such law could be more general, even though the most of time it is enough considering a linear relation as follows

$$\sigma_f(\alpha) = \sigma_e + H'\alpha \Longrightarrow \ d\sigma_f(\alpha) = H'd\alpha \tag{70}$$

where H' corresponds to the *hardening parameter* [4].
In the other hand, the elastoplastic modulus from equation 69 can be computed considering the additive decomposition of strain and elastoplastic regime for plastic loading

$$dF(\sigma, \alpha) = 0 \Longrightarrow \ d|\sigma| - d\sigma_f(\alpha) = 0 \Longrightarrow \ sign(\sigma)d\sigma - H'd\alpha = 0 \tag{71}$$

and substituting equation 66 ($d\alpha = sign(\sigma)d\varepsilon^p$)

$$sign(\sigma)d\sigma - sign(\sigma)H'd\varepsilon^p = 0 \Longrightarrow \ d\varepsilon^p = \frac{d\sigma}{H'} \tag{72}$$

establishing now the additive decomposition of strain and the last expression, then

$$d\varepsilon = d\varepsilon^e + d\varepsilon^p = \frac{d\sigma}{E} + \frac{d\sigma}{H'} = \left[\frac{1}{E} + \frac{1}{H'}\right] d\sigma$$

therefore

$$d\sigma = \frac{1}{\dfrac{1}{E} + \dfrac{1}{H'}} d\varepsilon \Longrightarrow \begin{cases} d\sigma = E^{ep} d\varepsilon \\ E^{ep} = \dfrac{EH'}{E + H'} \end{cases} \tag{73}$$

4.1.10 Failure Theories

Tresca's Yield Criterion

The Tresca's criteria is used for modelling the behavior of ductile materials, and establishes that elastic domain is over for a certain midpoint, when the maximum acting tangential stress at any plane is reached in τ_{max}, corresponding to the half of uniaxial elastic limit σ_y [4]

$$\tau_{max} = \frac{\sigma_1 - \sigma_3}{2} = \frac{\sigma_y}{2} \tag{74}$$

The next diagram represents the tensional state in three dimensions by the Mohr's circles, it describes how failure and the end of elastic domain gets closer as soon as the radius of the circle $(\sigma_1 - \sigma_3)$ reaches the horizontal line for $\tau = \tau_{max}$, and consequently, becoming tangent to it or being beyond the yield surface.

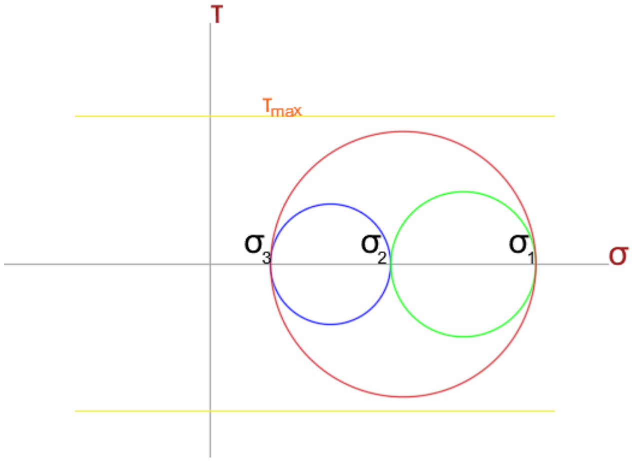


Figure 28: Three dimensional state with Mohr's circles.

Tresca's criteria might also be expressed as a function of the second and third invariants (J_2' and J_3'), without depending on first invariant I_1 [4]

$$F(\sigma) = F(J_2', J_3') = (\sigma_1 - \sigma_3) - \sigma_y = 0 \tag{75}$$

Von Mises Yield Criterion

The Von Mises criteria is adequate as failure theory for studying ductile materials. It is known that rupture is produced by tangential components of stress; and *hydrostatic stress*, either in tension or compression, have an elastic behavior [4].
The yielding surface is defined by the Von Mises criteria

$$F(\sigma) = \bar{\sigma}(\sigma) - \sigma_y = 0 \tag{76}$$

where

- $\bar{\sigma}(\sigma) = \sqrt{3J_2'}$ that represents *effective stress.*
- σ_y = yield stress.

An alternative expression for effective stress might be the following, it is in terms of the principal stresses and derived from computing the second invariant of J' [4]

$$F(\sigma) = \frac{1}{\sqrt{2}}\sqrt{[(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_1 - \sigma_3)^2]} - \sigma_y = 0 \tag{77}$$

The picture below shows two dimensional yield surfaces for $\sigma_2 = 0$. Let us observe that for each tensional state that remains within the cian contour can be considered as safe, and vice versa. Also, the circumscribed polygon (pink shape) on the Von Mises surface lets us realize that the use of this approach gives a more conservative result, and probably there could be estimated with certain margin of safety respect to Von Mises criterion. The maximum difference between the borders of both surfaces are approximately valued up to 15%.
It is precise to remember that stresses purely hydrostatic are associated with the change of volume for the body studied, and the tangential or *deviatoric* are associated with shape distortion and yield.

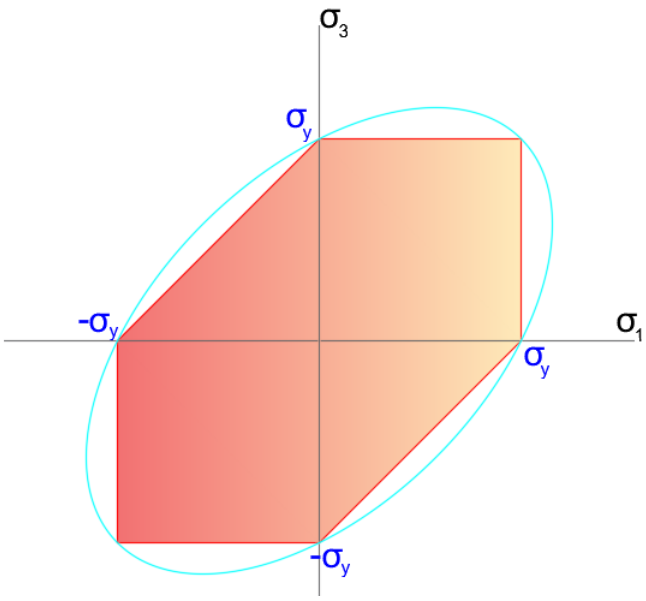


Figure 29: Von Mises (cian contour and white region) and Tresca (pink shape) yield surfaces.

4.2 Introduction to Finite Element Method

4.2.1 Background

Many of the physical phenomena in real life are rigorously described in terms of partial differential equations. The solution of the system of equations requires to be solved by classic analytical methods over complex domains. The *Finite Element Method* (FEM) consists of a numerical approaching employed in the engineering and scientific computation fields; the FEM is a method for solving physical problems in an extensive way. Such problems can be related to stress analysis, heat transfer, fluid flow and electromagnetics by computer simulation. Even nowadays, the FEM has already introduced in medicine and biomechanical engineering problems [8].
The *discretization* is the process where a determined continuum body is divided into smaller equivalent pieces (finite elements) interconnected each other by nodal points or nodes.
The modern development of FEM use arises in 1940 in the field of structural engineering with the work of by Hrennikoff in 1941 and McHenry in 1943, who used an array of one-dimensional elements for the solution of stresses in continuous solids. In a paper published in 1943 but not widely recognized, Courant proposed setting up the solution of stresses in a variational form, and after that, he introduced piecewise interpolation (also known as shape) functions over triangular entities or subregions making simpler the calculation process. In 1947 Levy developed the *flexibilities* or *forces method*, and in 1953 his work would suggest that another methods could be useful and promising for solving problems in structural analysis, such as *stiffness* or *displacements method*.
In 1954 Argyris and Kelsey developed matrix structural analysis methods using energy principles. This event would have carried the success of the FEM application for several problems. On the other hand, Tuner in 1960 had a first approaching in two-dimensional elements that later would lead to derivation of stiffness matrices for triangular and rectangular elements in plane stress as well as truss and beam elements. Along the development and progress of high-speed digital computing in early 1950s, the method was successfully introduced and adopted in its matrix notation [8].

4.2.2 Advantages of FEM

The FEM has been an applied for solving numerous problems, and beyond the orientation, it has become very popular due to the advantages and availabilities that offers, such as [8]:

- Modeling irregular geometries quite easily.
- Handle general load conditions without difficulty.
- Model bodies composed of several different materials because the elements equations are evaluated individually.
- Include dynamics effects.
- Vary the size of the elements to make it possible to use smaller elements when it is necessary (attending to the increasing in computational availability).
- Handle nonlinear behavior existing with large deformations and nonlinear materials.

4.2.3 Numerical Solutions

As discussed earlier, a very good accuracy and effectiveness in a solution built by FEM implies a very refined mesh. But as refined is such mesh, the computational cost also implies a higher demand of procedures to build the accuracy desired. In static analysis, it could be said that requirement of computing time, it is considerable lower respect to dynamic analysis. This depends on the appropriate use of constraints for equilibrium and the number of elements employed. Otherwise, if the bases to build the solution are badly used, then the computing time might be larger than expected and probably giving unreliable values [5].
Essentially, there are two different classes of methods employed for the solution of discrete systems with numerical approximations [5]:

- Direct solutions techniques. For this method, the equations are solved using a certain and limited number of steps and operations that are predetermined in an exact manner.
- Iterative solutions techniques. In this particular case, the solution is reached after repeating a determined process several times until reducing the estimated error considered as acceptable.

4.2.4 FEM Process

In the employment of finite element method, it results convenient having a sequential process that let us approach the solution of the mathematical model once all the information is gathered and organized. This process will make clear the steps involved for the use of the numerical tools and computer resources, and consequently, to have a consistent solution according to the predicted response. Below is shown an example of how organize the information and the stages for solving computational mechanics problems with the finite elements method.

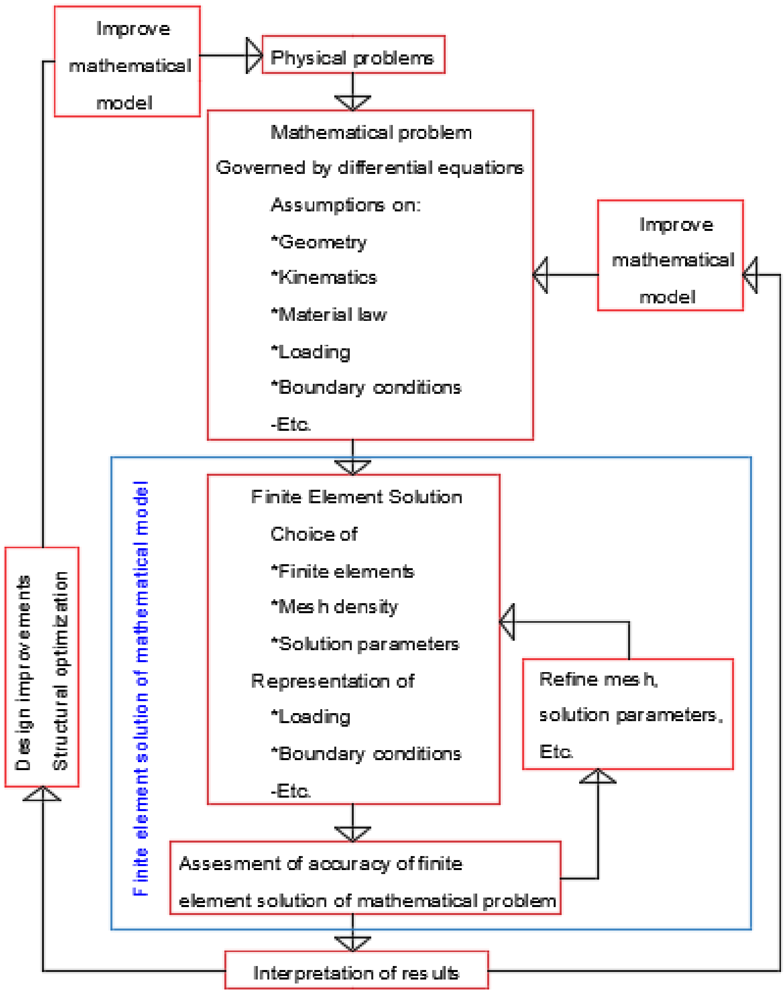


Figure 30: FEM process according to Klaus-Jürgen Bathe (Finite Element Procedures, 2014).

4.3 Finite Element Formulation and Discretization

4.3.1 Introduction

From the previous background in solids mechanics, the solution will be constructed carefully attending to the nature of the differential equations that govern the problem. However, as it was mentioned before, the approach by the *Strong Form* can barely give an analytic solution for few problems with oversimplified assumptions and simple domains. Besides, another problem is that mostly, it is almost impossible to prove the uniqueness, stability and existence for the solution desired [6]. Hence, the approach by the *Weak Form* is an alternative that at least can demonstrate existence, uniqueness and stability for the particular solution.
In general terms, there must be a function **u** such that it satisfies a certain differential equations set [11]

$$\mathcal{A}(u) = \begin{Bmatrix} \mathcal{A}_1(u) \\ \mathcal{A}_2(u) \\ \mathcal{A}_3(u) \\ . \\ . \\ . \\ \mathcal{A}_n(u) \end{Bmatrix} = 0 \tag{78}$$

over certain domain (volume, area, etc.), and considering the boundary conditions

$$\mathcal{B}(u) = \begin{Bmatrix} \mathcal{B}_1(u) \\ \mathcal{B}_2(u) \\ \mathcal{B}_3(u) \\ . \\ . \\ . \\ \mathcal{B}_n(u) \end{Bmatrix} = 0 \tag{79}$$

around the boundary, Γ, of the domain shown in Figure 31.

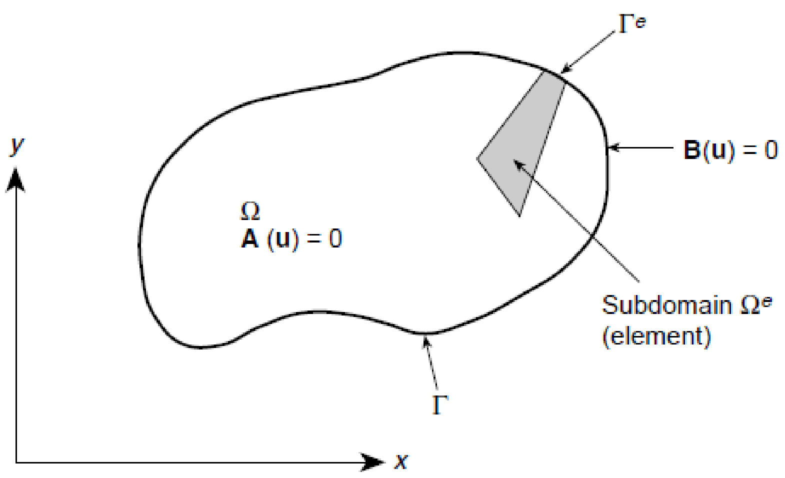


Figure 31: Domain Ω and boundary Γ (The Finite Element Method: Its Basis and Fundamentals, 2005).

To build the approach in the weak form, there will be necessary the use of *test functions* and *trial* *functions*. Then, the space where the values of test functions are acceptable is defined by [6]

$$\delta u_i(x) \in \mathcal{U}_0, \;\; \mathcal{U}_0 = \{\delta u_i \mid \delta u_i \in \mathcal{C}^0(x), \delta u_i = 0 \; in \; \Gamma^D\} \tag{80}$$

where $\mathcal{C}^0$ corresponds to the continuity of the function, and the boundary is defined by $\Gamma = \Gamma^D \bigcup \Gamma^N$. In the other hand, the space where the trial functions come from is defined by [6]

$$u_i(x,t) \in \mathcal{U}, \;\; \mathcal{U} = \{u_i \mid u_i \in \mathcal{C}^0(x), u_i = \bar{u_i} \; in \; \Gamma^D\} \tag{81}$$

4.3.2 Discretization

As it was earlier discussed the concept very briefly, the discretization consists of dividing the whole domain of study into subdomains or small pieces (*finite elements*), in order to get an approximate solution instead of the analytical solution. Once analyzed the entity, the exact solution will be found for all the nodes located on the domain, and an interpolation is the result for those that are found out of the mesh but within of the continuum. As long as the mesh is more refined, then the solution will tend to increase the accuracy, but it also increases the computational cost.
Let us suppose that the domain Ω is discretized with enough finite elements, and then they constitute the mesh. For each finite element on the mesh, the displacements field is approximated by [6]

$$u_i^h(x,t) = \sum_{I=1}^{n_{nodes}} N_I(x) u_{iI}(t) \quad \forall i = 1, \; n_{dim} \tag{82}$$

where $N_I(x)$ are the *shape functions* for each node I, n_{nodes} is the number of nodes on the mesh of the problem, and $u_{iI}(t)$ are the value for displacement in each node I in the i global direction, i. e.

$$u^h(x,t) = \sum_{I=1}^{n_{nodes}} N_I(x) u_I(t) \tag{83}$$

$$v^h(x,t) = \sum_{I=1}^{n_{nodes}} N_I(x) v_I(t) \tag{84}$$

$$w^h(x,t) = \sum_{I=1}^{n_{nodes}} N_I(x) w_I(t) \tag{85}$$

From the infinitesimal strain tensor (Equation 45), it can be rewritten in order of the subscripts kl instead of ij for convenience as

$$\varepsilon_{kl} = \frac{1}{2}\left(\frac{\partial u_k}{\partial X_l} + \frac{\partial u_l}{\partial X_k}\right) = \frac{1}{2}\left(\frac{\partial u_i}{\partial X_l}\delta_{ik} + \frac{\partial u_i}{\partial X_k}\delta_{il}\right) \tag{86}$$

Then, if the displacements field u_i are approximated for u_i^h, replacing Equation 82 in 86

$$\varepsilon_{kl}^h = \sum_{I=1}^{n_{nodes}} \frac{1}{2}\left(\frac{\partial N_I(x)}{\partial x_l}\delta_{ik} + \frac{\partial N_I(x)}{\partial x_K}\delta_{il}\right) u_{iI}(t) \quad \forall i = 1, n_{dim} \tag{87}$$

Defining the next fourth order tensor as the strain-displacement tensor, then

$$B_{iklI} = \frac{1}{2}\left(\frac{\partial N_I(x)}{\partial x_l}\delta_{ik} + \frac{\partial N_I(x)}{\partial x_K}\delta_{il}\right) \tag{88}$$

Hence, the infinitesimal strain can be expressed as follows

$$\varepsilon_{kl}^h = \sum_{I=1}^{n_{nodes}} B_{iklI} u_{iI} \quad \forall i = 1, n_{dim} \tag{89}$$

Analogously, developing the discretization for the virtual strain, the expression obtained is

$$\delta\varepsilon_{kl}^h = \sum_{I=1}^{n_{nodes}} B_{iklI} \delta u_{iI} \quad \forall i = 1, n_{dim} \tag{90}$$

The internal virtual work generated in the body of study can be described for

$$\delta W^{int} = \sum_{I=1}^{n_{nodes}} \delta u_{iI} f_{iI}^{int} = \int_{\Omega} \delta\varepsilon_{kl}^{h} \sigma_{kl} \; d\Omega \quad \forall i = 1, n_{dim} \tag{91}$$

and substituting the Equation 90, then

$$\delta W^{int} = \sum_{I=1}^{n_{nodes}} \delta u_{iI} f_{iI}^{int} = \sum_{I=1}^{n_{nodes}} \delta u_{iI} \int_{\Omega} B_{iklI} \sigma_{kl} \; d\Omega \quad \forall i = 1, n_{dim} \tag{92}$$

In the case of the external virtual work, the expression is

$$\delta W^{ext} = \sum_{I=1}^{n_{nodes}} \delta u_{iI} f_{iI}^{ext} = \sum_{I=1}^{n_{nodes}} \delta u_{iI} \left[\int_{\Omega} \rho \; b_{iI} \; d\Omega + \int_{\Gamma^N} t_{iI} d\Gamma \right] \quad \forall i = 1, n_{dim} \tag{93}$$

It is important mention that if the shape functions are used for the discretization of the test and trial functions, therefore, the approach built by the weak form is known as *the Galerkin method* [6] or also it is known as *the method of weighted residuals.* The problem can also achieve a solution determining *variational functionals* for which stationarity is sought [11].

4.3.3 Three-Dimensional Elements

The two basic categories of three-dimensional elements are hexahedral and tetrahedral elements. The former are generalizations of an assembly of quadrilateral elements, and similarly with the generalizations of the assembly of triangles for the tetrahedral elements [12]. The basic lower order for three-dimensional finite elements are the eight-node hexahedral and four-node tetrahedral, even though higher-order configurations can be structured.

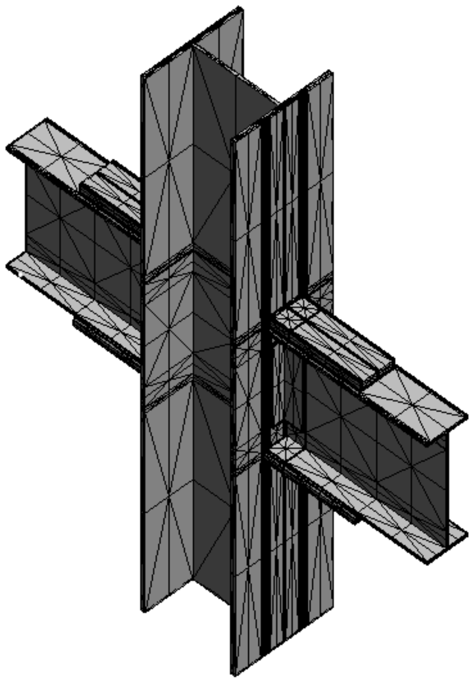


Figure 32: Mesh generated trough the employment of tetrahedral elements (11,787 nodes and 52,560 elements). Image obtained from pre process of the interface on Software **GiD**. The picture shown depicts the connection model to be studied and validated (with a set of angles).

Hexahedral Elements

The eight-node hexahedral finite element can be described attending to the position of its nodes in an *isoparametric* space with coordinates ξ, η and ζ. The map to the physical domain is

$$x(\xi,\eta,\zeta) = N^{8H}(\xi,\eta,\zeta)x^e \tag{94}$$

$$y(\xi,\eta,\zeta) = N^{8H}(\xi,\eta,\zeta)y^e \tag{95}$$

$$z(\xi,\eta,\zeta) = N^{8H}(\xi,\eta,\zeta)z^e \tag{96}$$

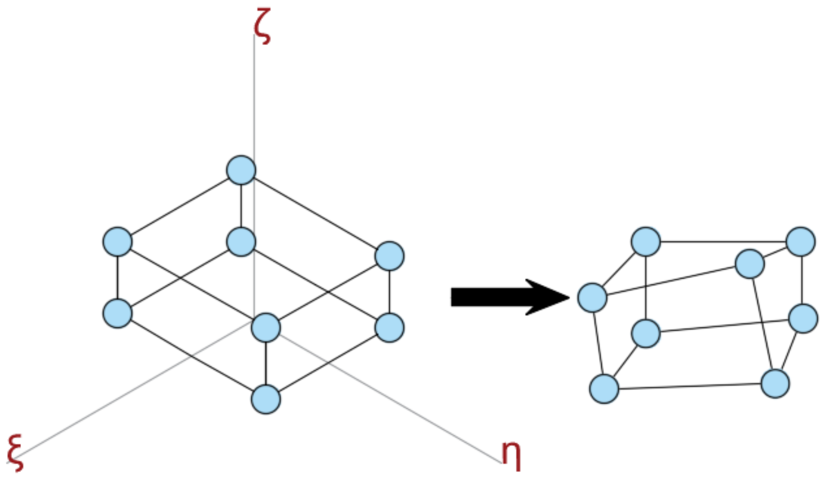


Figure 33: Mapping of the eight-node hexahedral finite element (initial configuration) and its deformed configuration.

The eight-node hexahedral shape functions can be constructed by a tensor of one-dimensional linear shape functions as [12]

$$N^{8H}(\xi,\eta,\zeta) = N_I^{2L}(\xi)N_J^{2L}(\eta)N_K^{2L}(\zeta) \tag{97}$$

The Jacobian Matrix $\mathcal{J}$ in three dimensions is

$$\mathcal{J} = \begin{bmatrix} \frac{\partial x}{\partial \xi} & \frac{\partial y}{\partial \xi} & \frac{\partial z}{\partial \xi} \\ \frac{\partial x}{\partial \eta} & \frac{\partial y}{\partial \eta} & \frac{\partial z}{\partial \eta} \\ \frac{\partial x}{\partial \zeta} & \frac{\partial y}{\partial \zeta} & \frac{\partial z}{\partial \zeta} \end{bmatrix} \tag{98}$$

The resultant integral over the subdomain can be expressed by

$$\int_\Omega f(\xi,\eta,\zeta) = \int_{\xi=-1}^{\xi=1}\int_{\eta=-1}^{\eta=1}\int_{\zeta=-1}^{\zeta=1} ||\mathcal{J}||f(\xi,\eta,\zeta)\; d\xi\; d\eta\; d\zeta = \sum_{i=1}^{n_{gp}}\sum_{j=1}^{n_{gp}}\sum_{k=1}^{n_{gp}} W_iW_jW_k||\mathcal{J}||f(\xi,\eta,\zeta) \tag{99}$$

Tetrahedral Elements

Studying the next tetrahedral parent and physical domain illustrated below, the tetrahedral coordinates can define the total volume as follows: at any belonging point **P** over the entity (Figure 34) split the initial body Ω into four tetrahedrons. Therefore, the volume coordinates are described in the next form:

$$\xi_1 = \frac{Volume_{P234}}{\Omega^e} \tag{100}$$

$$\xi_2 = \frac{Volume_{P134}}{\Omega^e} \tag{101}$$

$$\xi_3 = \frac{Volume_{P124}}{\Omega^e} \tag{102}$$

$$\xi_4 = \frac{Volume_{P123}}{\Omega^e} \tag{103}$$

Noticing that:

$$\xi_1 + \xi_2 + \xi_3 + \xi_4 = 1 \tag{104}$$

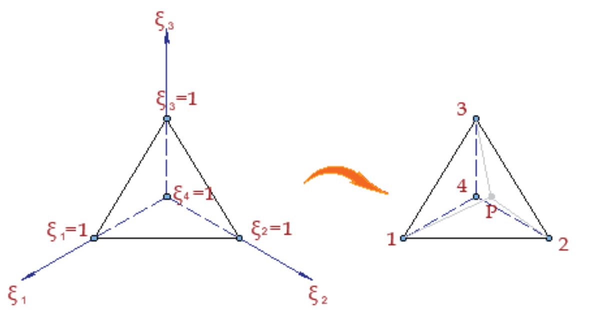


Figure 34: Mapping of the four-node tetrahedron from left the parent to right the physical Cartesian coordinate system. Also, it is shown the interior point P (not a node) in physical domain.

Integration order	**Degree of precision**	**Weights**	ξ_1	ξ_2	ξ_3
One-point	2	1.00	0.25	0.25	0.25
Four-point	3	0.25	0.58541020	0.13819660	0.13819660
		0.25	0.13819660	0.58541020	0.13819660
		0.25	0.13819660	0.13819660	0.58541020
		0.25	0.13819660	0.13819660	0.13819660
Five-point	4	-0.80	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$
		0.45	$\frac{1}{3}$	$\frac{1}{6}$	$\frac{1}{6}$
		0.45	$\frac{1}{6}$	$\frac{1}{3}$	$\frac{1}{6}$
		0.45	$\frac{1}{6}$	$\frac{1}{6}$	$\frac{1}{3}$
		0.45	$\frac{1}{6}$	$\frac{1}{6}$	$\frac{1}{6}$

Gauss quadrature weights and points for tetrahedral elements.

5 Research Methodology, Computing and Results

5.1 Introduction

This research project is well defined in the abstract, noticing the important mention of computational advances and the expansive quantity of technological resources for the application of numerical solutions in structural engineering field.
Besides, the complexity of engineering problems consequently will change as the initial conditions do. Numerical analysis results an interesting discipline to solve approximately the chaotic and mysterious nature around the phenomena that human being could explain.
After studying the mathematical and physical axioms that describes motion and the conservation of energy, the unknowns and results from differential equations must have meaning according to the initial conditions given from the problem data. Lately, having the knowledge of the structural analysis, the engineer have the duty to adapt the information with the advances in technology if success want to be reached, and also having the scope or limitations given the case.

5.1.1 Motivation, Objectives and Justification

Motivation and Justification

Science and engineering are in the mid of a constant evolution provided by scientific attributions and using the technological resources. In civil engineering field of study, and focusing on the structural engineering in a particular way, the present work developed hopes for contributing according to the next sentences:

- Implementation and employment of advanced computational tools that ease solving problems of solid mechanics and structural engineering.
- Providing configurations and realistic solutions that satisfy requirements about strength and safety, besides, that could be easily taken to practice (real construction).
- Optimization of configurations and computation time for designing.
- Modelling of complex geometries.
- Comparing computing methods and evaluating structural reliability in qualitative or quantitative terms.

Objectives

The goals pretended for the present investigation involves:

- Creating computational tools that might help studying multiple models, making easy the process on creating geometries and applying boundary conditions.
- Evaluating the strength and safety conditions based on the AISC-360-16, and consequently, studying either qualitative or quantitative parameters that could have influence on the structural performance for any such particular building.
- Studying and comparing the conditions related to nominal strength for a pre-qualified rigid steel connection with shear plates, respect to the prototype using angles (Figure 43).
- Validating and supporting the design hypothesis and methodologies of computation.
- Trough information obtained, support and validate results to get the knowledge of atypical features in performance for non-qualified connection as a potential solution that guarantees safety, strength and reliability; and therefore, consider it as qualified.
- Studying the structural efficiency of welds.

5.2 Resources

5.2.1 GiD: Preprocess and Postprocess

GiD is a powerful pre and post processor developed by the International Numerical Methods Center (*CIMNE*) with the purpose of translate the complexity of engineering and scientific problems trough computer employment and all the tools involved.

Figure 35: GiD Software's logo.

As engineer, the Computer Aided Drawing (*CAD*) would be one the first and, maybe the few options about how provide a three-dimensional model or geometry of study. But, this tool and all the set of skills in CAD would start to reach limitations in providing geometries according to the next premises

- The interface destined for CAD inside GiD is not that skillful manipulating commands to generate geometries compared to another programs.
- The software allows importing files originated from external CAD programs, but, sometimes, the geometries imported cause problems in the nodes; due to a lack of continuity or inconsistencies in boundary conditions that the software can not prevent by itself. A clear example of the consequences related to last comments, might be explained in how the solver can give back meaningless results, compared to the expected or "rational" performance predicted obeying physics laws and closeness with the "expected performance" for all that can understand the nature of mechanics.
- Changes in project are a constant threat to the previous work and effort invested on projects. As initial configuration changes, also conditions do, and time or limited resources might turn controversial with possible failures or overrun.
- Still having a friendly software that allows studying models of steel connections, the limitations could appear when a certain big number of configurations are demanded to be calculated for an original project. To recreate a new whole model could be negatively reflected in delivering times and scope of the project, or whatever the consequence is given to the case.

The advantage that lies on the use of GiD respect to any other software, consists of the interface manipulation by uploading programming codes with the object of employing carefully, and a simpler manner; the whole information from input and output files. GiD offers the following customization features:

- Complete menu's can be customized and created to suit the specific needs of the user's simulation software.
- Simple interfaces can be developed between the data definition and the simulation software.
- Simple interfaces based on scalar, vector and matrix quantities can be developed for the results visualization.

The customization in GiD is done by creating a **Problem Type**.

Problem Type Structure

A *problem type* might be defined as a files collection that allows the user to stablish the input parameters involved for a specific computational problem [13]. This ease to customize the interface can be more simplified according respect to the newer versions. The success compared to older versions come from:

- The hierarchical system and features of XML that allows saving information efficiently.
- The ease to construct *standardized windows* by programming oriented to objects from *data tree structures* with the purpose of input data classification.
- The facility to couple entities with identical properties by groups.
- The assignation of boundary conditions and other properties can be simpler to modify.

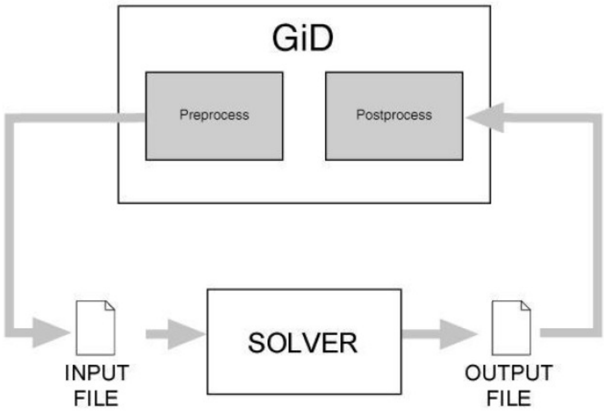
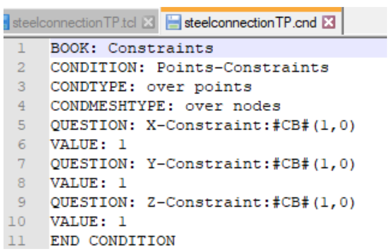


Figure 36: GiD scheme operation process (GiD 13: Customization manual [13]).

GiD has been designed to be a general-purpose Pre- and Postprocessor; consequently, the configurations for different analyses must be performed according to the particular specifications of each solver [13]. It is therefore necessary to create specific data input files for every solver. However, GiD lets you perform this configuration process inside the program itself, without any change in the solver, and without having to program any independent utility.
The files collection is composed of:

- **ProblemType.cnd** (**CND extension**):
  This file contains the information related to boundary conditions in the different types of entities (as lines, surfaces or volumes). For a typical structural analysis problem, the example can be clear as the configuration of displacement restrictions, initial stresses/displacements, loads, and so on. The file is not pretended to be modified, and such containing is not that necessary for running the problem type code.




```
BOOK: Constraints
CONDITION: Points-Constraints
CONDTYPE: over points
CONDMESHTYPE: over nodes
QUESTION: X-Constraint:#CB#(1,0)
VALUE: 1
QUESTION: Y-Constraint:#CB#(1,0)
VALUE: 1
QUESTION: Z-Constraint:#CB#(1,0)
VALUE: 1
END CONDITION
```

Figure 37: CND extension file.

- **ProblemType.xml** (**XML extension**):
  This is configured just once, in the order that it is just merely informative. It declares information related to the configuration of the problem type, such name, version, file browser icon, password validation or message catalogue location, history news, etc. But, still the short size saved inside, the existence and adequate redaction of this results essential. Not even a single character can be ignored or dismissed, as the identifiers so.
  As mentioned before, this precaution is also important for the denomination of the archive. Any alteration along the files names could not let run the expected sequence of the program, and probably get difficulties with process.

```
steelconnectionTP.tcl  steelconnectionTP.cnd  steelconnectionTP.xml
<?xml version="1.0" encoding="utf-8"?>

<Infoproblemtype version="1.0">
  <Program>
    <Name>steelconnectionTP</Name>
    <Version>1.0</Version>
    <MinimumGiDVersion>12.1.11d</MinimumGiDVersion>
    <CustomLibAutomatic>1</CustomLibAutomatic>
  </Program>
</Infoproblemtype>
```

Figure 38: XML extension file.

- **ProblemType.spd** (**SPD extension**):
  Based on XML, this is responsible for initial configuration of "data tree", and that includes the whole structured information for contour conditions, materials, etc. The alias of *data tree* arises from how the structure classifies information in several "branches", and grouping the field objects inside others.
  With XML, fields or tags can be created to customize the virtual interaction with user (menus, fields, etcetera) and for saving data.

```
steelconnectionTP.tcl  steelconnectionTP.cnd  steelconnectionTP.xml  steelconnectionTP_default.spd
<?xml version="1.0" encoding="utf-8"?>
<steelconnectionTP_data show_menubutton_about="0" version='1.0'>
<container n="column" pn="Connection column" help="Geometric properties of the column" actualize_tree="1">
    <value n="bfcol" pn="flange" v="36.83" help="flange dimension" actualize_tree="1"/>
    <value n="hcol" pn="height" v="35.56" help="height dimension" actualize_tree="1"/>
    <value n="tfcol" pn="flange thickness" v="1.80" help="flange thickness dimension" actualize_tree="1"/>
    <value n="twcol" pn="web thickness" v="1.12" help="web thickness dimension" actualize_tree="1"/>
    <value n="ht" pn="height column" v="200.0" help="define height of the column" actualize_tree="1"/>
</container>
```

Figure 39: SPD extension file.

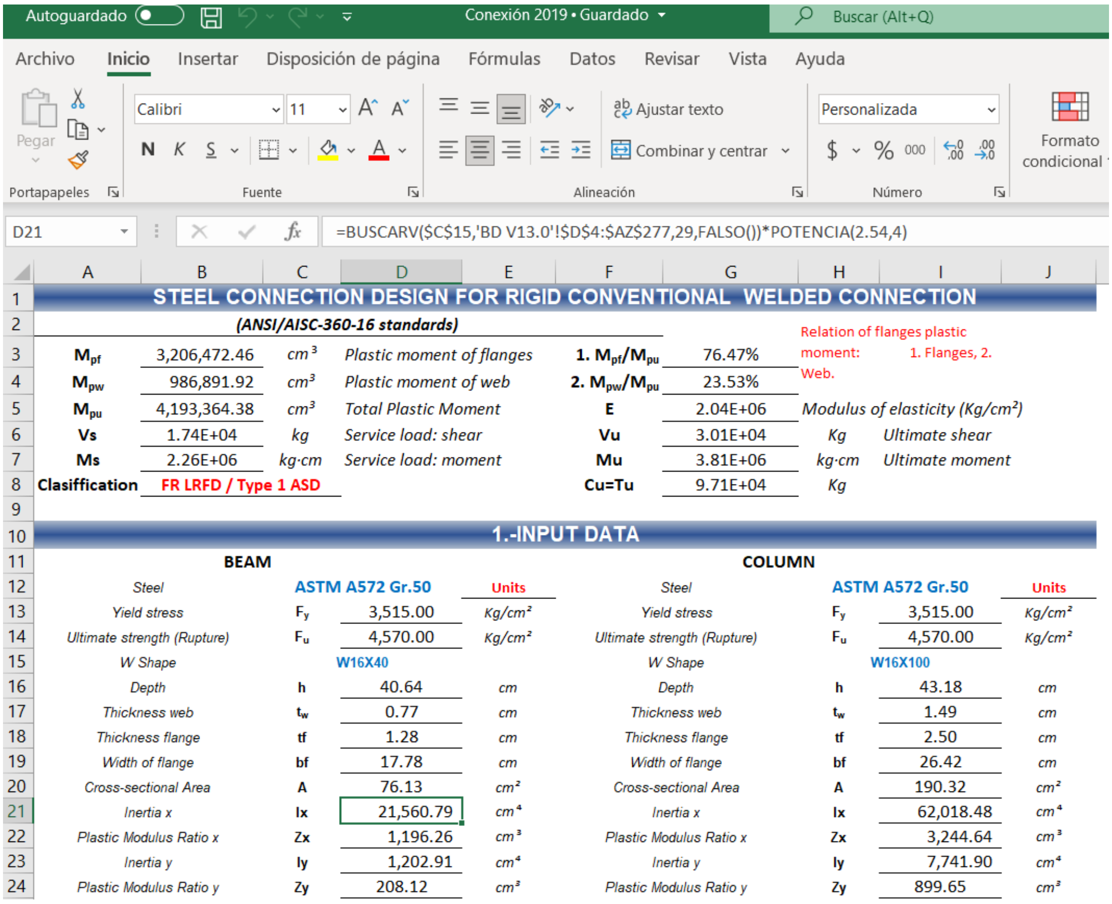
- **ProblemType.tcl** (**TCL extension**):
  This is absolutely coded in TCL/Tk programming language, and for the thesis scope, just the employed terms or commands will be briefly described later as the order of importance and the linked roll for solution processes.

```
steelconnectionTP.tcl
proc steelconnectionTP::DrawShearplates { } {
    set rootXML [customlib::GetBaseRoot]
    variable weldsp [get_domnode_attribute [$rootXML selectNodes "\container\[@n='shearplate'\]/value\[@n='twsp'\]"] v]
    variable thsp [get_domnode_attribute [$rootXML selectNodes "\container\[@n='shearplate'\]/value\[@n='tps'\]"] v]
    variable hc [get_domnode_attribute [$rootXML selectNodes "\container\[@n='column'\]/value\[@n='hcol'\]"] v]
    variable bfc [get_domnode_attribute [$rootXML selectNodes "\container\[@n='column'\]/value\[@n='bfcol'\]"] v]
    variable twb [get_domnode_attribute [$rootXML selectNodes "\container\[@n='beam'\]/value\[@n='twbeam'\]"] v]
    variable heightcol [get_domnode_attribute [$rootXML selectNodes "\container\[@n='column'\]/value\[@n='ht'\]"] v]
    variable lsp [get_domnode_attribute [$rootXML selectNodes "\container\[@n='shearplate'\]/value\[@n='lsp'\]"] v]
    variable dsp [get_domnode_attribute [$rootXML selectNodes "\container\[@n='shearplate'\]/value\[@n='bsp'\]"] v]
```

Figure 40: TCL extension file.

5.2.2 VBA Excel Tools

A Microsoft Excel Macros-Enabled Workbook was created as a first step to approximate the dimension parameters that probably could guarantee strength and safety, and consequently, to recreate the computational models with a very detailed and refined geometry provided by the structured code of problem types. This particular workbook has linked a data base referent to the whole system of steel profiles from *AISC (BD V13.0)*.
Besides, the spreadsheets contained were featured and personalized with functions to estimate strength and diverse properties, with the purpose of optimizing time for calculations and to ease the interaction to work several configurations in multiple cases i.e. changes on project that could demand an urgent response.



	A	B	C	D	E	F	G	H	I	J
1	STEEL CONNECTION DESIGN FOR RIGID CONVENTIONAL WELDED CONNECTION									
2			(ANSI/AISC-360-16 standards)							
3	M_{pf}	3,206,472.46	cm³	Plastic moment of flanges		1. M_{pf}/M_{pu}	76.47%	Relation of flanges plastic moment: 1. Flanges, 2. Web.		
4	M_{pw}	986,891.92	cm³	Plastic moment of web		2. M_{pw}/M_{pu}	23.53%			
5	M_{pu}	4,193,364.38	cm³	Total Plastic Moment		E	2.04E+06	Modulus of elasticity (Kg/cm²)		
6	Vs	1.74E+04	kg	Service load: shear		Vu	3.01E+04	Kg	Ultimate shear	
7	Ms	2.26E+06	kg·cm	Service load: moment		Mu	3.81E+06	kg·cm	Ultimate moment	
8	Clasiffication	FR LRFD / Type 1 ASD				Cu=Tu	9.71E+04	Kg		
9										
10	1.-INPUT DATA									
11		BEAM					COLUMN			
12		Steel	ASTM A572 Gr.50		Units	Steel		ASTM A572 Gr.50		Units
13		Yield stress	F_y	3,515.00	Kg/cm²	Yield stress		F_y	3,515.00	Kg/cm²
14		Ultimate strength (Rupture)	F_u	4,570.00	Kg/cm²	Ultimate strength (Rupture)		F_u	4,570.00	Kg/cm²
15		W Shape	W16X40			W Shape		W16X100		
16		Depth	h	40.64	cm	Depth		h	43.18	cm
17		Thickness web	t_w	0.77	cm	Thickness web		t_w	1.49	cm
18		Thickness flange	tf	1.28	cm	Thickness flange		tf	2.50	cm
19		Width of flange	bf	17.78	cm	Width of flange		bf	26.42	cm
20		Cross-sectional Area	A	76.13	cm²	Cross-sectional Area		A	190.32	cm²
21		Inertia x	Ix	21,560.79	cm⁴	Inertia x		Ix	62,018.48	cm⁴
22		Plastic Modulus Ratio x	Zx	1,196.26	cm³	Plastic Modulus Ratio x		Zx	3,244.64	cm³
23		Inertia y	Iy	1,202.91	cm⁴	Inertia y		Iy	7,741.90	cm⁴
24		Plastic Modulus Ratio y	Zy	208.12	cm³	Plastic Modulus Ratio y		Zy	899.65	cm³

Figure 41: Overview of personalized Macros-Enabled spreadsheets with functions.

The premises of design and philosophy is taken from chapters 2 and 3 from this document, and also including the corresponding bibliographic references. Also, the spreadsheets own some warnings or advises programmed to be shown for user in all cases that conditions might provide low resistance or some procedures could not been replicated in reality as dimensions or features were studied previously.
The units are strictly fixed, the functions make the unit conversions from English system to the shown in Figure 41. At the end of every spreadsheet, and as long as the all parameters were carefully computed and studied, it shows a resume of union elements, dimensions, properties, steel sections, welding parameters, etc. The resume also has diagrams that link every parameter with the model to construct, location, assemblage, and another specifications.

5.3 Steel Connections Design using VBA Excel Spreadsheets and Finite Element Method

5.3.1 Approaching and Input Data

As it was previously discussed, the spreadsheets are based on premises and hypothesis from chapters 2 and 3, also, considering the philosophy of design (*ANSI/AISC 360-16*). The expressions for nominal strength and the correspondent requirements will provide a first geometry approximated that could accomplish determined demand. The dimensions of elements are concluded from computing as potential solution and having the criteria and knowledge from user. The units are particularly settled for every computational tool, and visually shown to inform the user about. The methodology proposed to study the unconventional or not qualified connection will be explained later ahead.
In different circumstances, the finite element method will be implemented trough the geometries taken from the approximations with the spreadsheets.
The models proposed to study for this research are seized from real constructions with metallic shapes and concrete slab, the particular descriptions with features are enlisted below:

- **MODEL 1:** Offices building.
  - Surface (approx): $400m^2$.
  - Levels: 2.
  - Classification of Seismic Region: $B - II$.

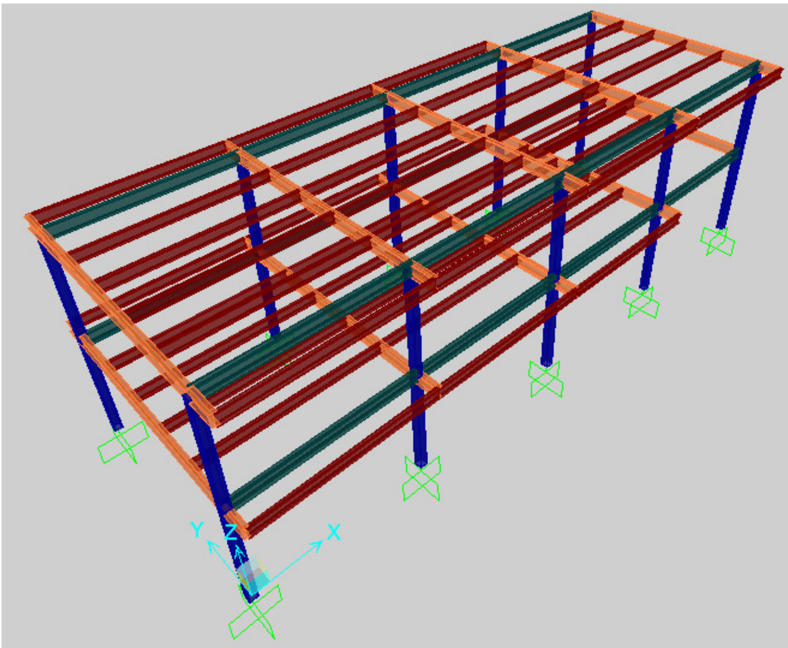


Figure 42: Extruded model of Offices building (MODEL 1,*Program: SAP2000*).

- **MODEL 2:** Library building.
  - Surface (approx): $4,000m^2$.
  - Levels: 3.
  - Classification of Seismic Region: $B - II$.

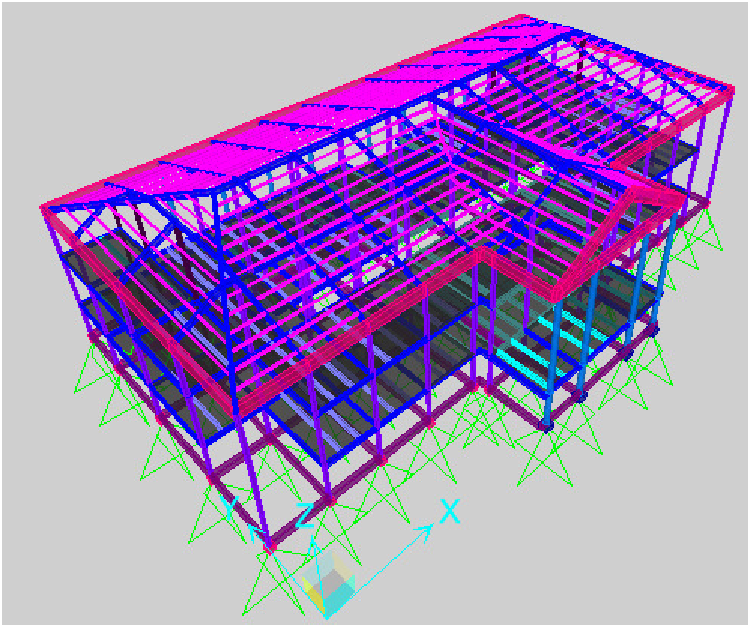


Figure 43: Extruded model of Library building (MODEL 2,*Program: SAP2000*).

Load Analysis

- **MODEL 1:** Steel Connection for Office Building.

Description	Designation: Dead Load ($D_L, kg/m^2$)
Concrete Slab	280.00
Installations	40.00
Total D_L	320.00

Description	Designation: Live Load ($L_L, kg/m^2$)
Maximum Live Load (L_m)	250.00
Maximum Instantaneous Live Load (L_i)	180.00

- **MODEL 2:** Steel Connection for a Library Building.

Description	Dead Load ($D_L, kg/m^2$)
Concrete Slab	280.00
Installations	40.00
Total D_L	320.00

Description	Designation: Live Load ($L_L, kg/m^2$)
Maximum Live Load (L_m)	350.00
Maximum Instantaneous Live Load (L_i)	250.00

Load Combinations and Patterns

The patterns of loads are settled as:

- P_p: Self weight of the structure.
- D_L: Dead load.
- L_L: Live load.
- L_i: Instantaneous live load.
- S_x: Seismic actions along "x" axis.
- S_y: Seismic actions along "y" axis.

The load combinations proposed to study the models represents the sum of the patterns involved multiplied for a scalar, i.e.

- DS-01= $P_p + D_L$
- DS-02= $1.3DS - 01 + 1.5L_L$
- DS-03= $1.1DS - 01 + 1.1L_L + 0.33S_y + 1.1S_x$
- DS-04= $1.1DS - 01 + 1.1L_L + 1.1S_y + 0.33S_x$
- SERV-01= $DS - 01 + L_i$
- SERV-02= $SERV - 01 + S_y + 0.3S_x$
- SERV-03= $SERV - 01 + 0.3S_y + S_x$

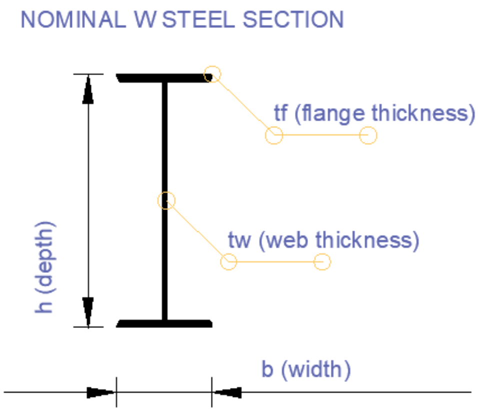


Figure 44: Dimensional features for a typical W steel profile.

These two steel connections will be studied within another 2 arbitrary models (MODEL 0 and MODEL 3), proposed to extrapolate output data with the purpose of encompassing the tens of W shapes not included for this research (conservative considerations). The group of connections will be compared using union plates for designations ASTM A36 and A572 Gr. 50, and simultaneously compared with its own optimized configuration.

Description of Configurations

The steel connections will be built with W standard shapes for beam-column domains, and the profiles own the next characteristics:

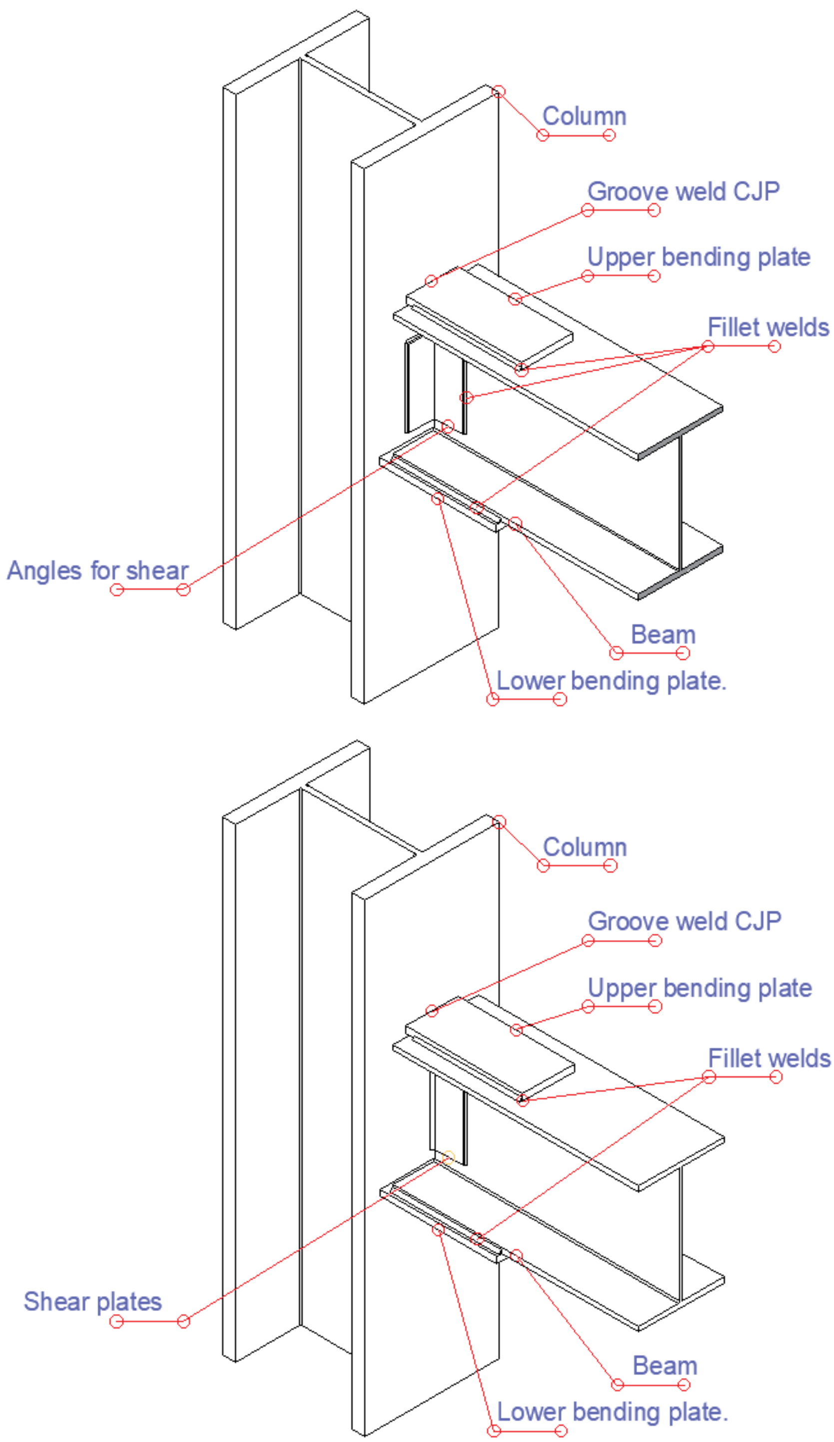


Figure 45: Geometric parameters of elements employed in beam-column connections, both models are considered in the approaching of solution by VBA spreadsheets and model for finite element analysis.

5.3.2 Review of Rigid Steel Connections (section 3.5) and Premises for Design

To make clear the context and being specifically straight to research purposes and study the prototypes, there must be information to realize at first instance, and that could associate the logical and congruent output information or results.
The geometrical specifications will be detailed further, but, taking a look of an abstract concept like representing an acting force is a complex dilemma. The elements described in problem types and spreadsheets consider a uniform distribution of the force acting in direction to the face with the purpose of recreate the bending moment as Figure 20 shows. Also, the shear will be considered in that way for the geometry on GiD surfaces generated, along the large one of the shear plate. For GiD, these uniform forces will be distributed by a computational algorithm over the nodes to solve numeric system of equations.
The spreadsheets oversimplify the abstraction of mechanics over the connection, and will only attend to accomplish conservatively the next limit states:

- For upper plate:
  - Yielding.
  - Fracture.
  - Strength of fillet welds.
  - Strength of groove weld.
- For shear plates:
  - Yielding.
  - Fracture.
  - Strength of fillet welds.
- For lower plate:
  - Strength of member considered as a column.

Let us comment that the previous requirements will apply for both models, but following a difference that lies on distribution of plastic moment over the cross-sectional beam member and union elements. If its observed, the contribution ratio of plastic modulus of flange and web, dividing it regarding to the entire section, this figure will oscillate about 70-75%. In other words, the missing part that completes plastic moment is carried or supposed to be absorbed by the web.
As the expressions shows

$$PMr_f = \frac{M_{pf}}{PM_S} = \frac{M_{pf}}{Z_x} \approx 75\%$$
$$PMr_w = \frac{M_{pw}}{PM_S} = \frac{M_{pw}}{Z_x} \approx 25\%$$

The next terminology relates to the further sub-chapters:

- QLF: makes reference to qualified model.
- $N - QLF$: makes reference to non-qualified model.
- STF: if the web column is stiffened.
- UNS: for all unstiffened configurations.
- PMr_f: plastic moment ratio generated for the flanges.
- PMr_w: plastic moment ratio generated for the web.
- PM_S: plastic moment modulus for cross-sectional area (Z_x).
- $A36/A572GR50$: designation of steel for joint plates.

The methodology followed is that for non-qualified configurations will work about to 70-75% of plastic moment due to the proportional contribution of flanges, and carrying on the calculations in the way of accomplishing with strength enough for welds and base metals. These must be designed to resist stresses: shear and normal due mainly for bending. The methodology considers (as hypothesis) that shear can be taken for the 65-70% of the fillet welds that join angles with the flange of the column for each non-qualified prototype. The missing plastic moment percentage is supposed to be resisted by the hatched area on the set of angles, let us take a view to the Figure 44.

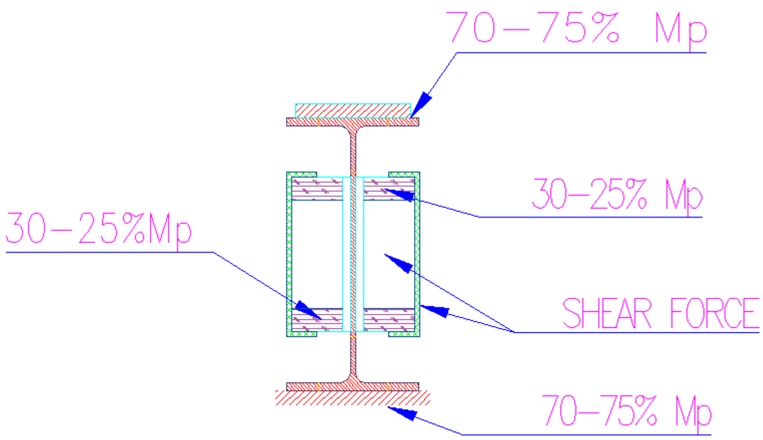


Figure 46: Forces and surfaces affected according to premises for non-qualified steel connections.

As mentioned before, the beam-column will be connected as diagram illustrates, the qualified archetype is joined by shear plates, but the non-qualified trough angles. The both pair of joints will be unified with welds.

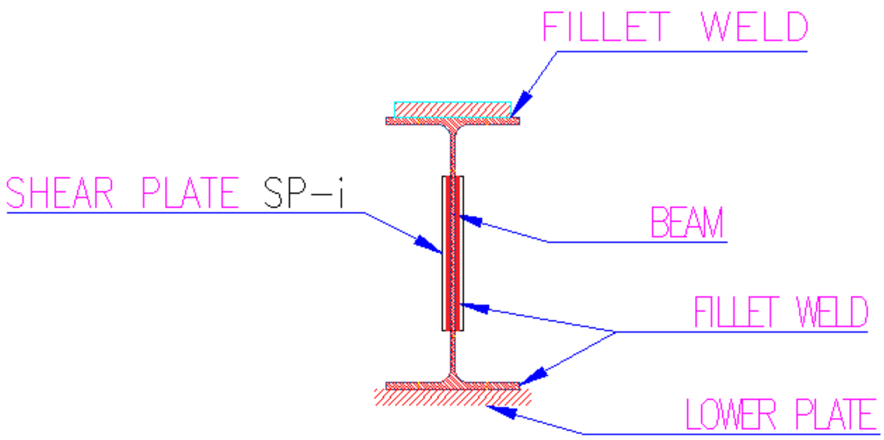


Figure 47: Joint elements for rigid qualified model.

Besides, the conventional connections will be tested as if 100% of the forces that produce bending moment is taken by the bending plates, and shear force by a couple of plates joined to the web of the beam. This, as traditional proceeding of calculating, (*and according to chapter 2, 3 from document*).
The conventional scheme sometimes provides a very thick plate that, in practice, is difficultly welded to the rest of elements. And then, along the research and comparison between models and methodologies, will be prioritized the optimization of bending plates with joint elements; and

consequently, studying further more about performance and strength in several conditions beyond already exposed. Still is important making sure about strength provided by welds and following the correspondent requirements.
Subsequently, besides the particularities and details already manifested before, there are more considerations in working the prototypes, the methodology followed and significant information that will sustain the succeed of reaching or dismissing objectives, according to the congruence of results.
The next premises constructs the system of the "problem types" coded for geometries, and consequently for the solver used, etc.

About physical properties:

- The Young's Modulus considered is $E = 2'040,000kg/cm^2$ *or* $(29,000ksi)$.
- The Poisson's Modulus employed is $\nu = 0.20$.
- The effect of weight or body forces of connection is considered within all load patterns and cases (density: $\rho = 7800kg/m^3$) to the structural analysis on *SAP2000*. But for finite element analysis and VBA spreadsheets, the weight consequences are neglected.
- The Elasticity or Young Modulus for welds, will be considered equal to the steel; attending to the relative small dimension for leg size of welds compared to the areas occupied by them and for dimensions belonging to the other elements.

About boundary conditions (loads and restrictions):

- The methodology proposed and described for non-qualified models will exclusively govern the design and dimensions for elements to generate in GiD. The qualified models with bending plates welded will consider the couple of forces provided by plastic moment on section at a 100%, and shear transference by the couple of thin plates welded to the web beam.
- The purple hatch on Figure 44 will be represented on finite element software analysis as an equivalent pair of forces linearly distributed along the width of angles welded to the flange of column. The tributary load area of the legs welded refers about the 25% to generate the plastic moment correspondent of web area.

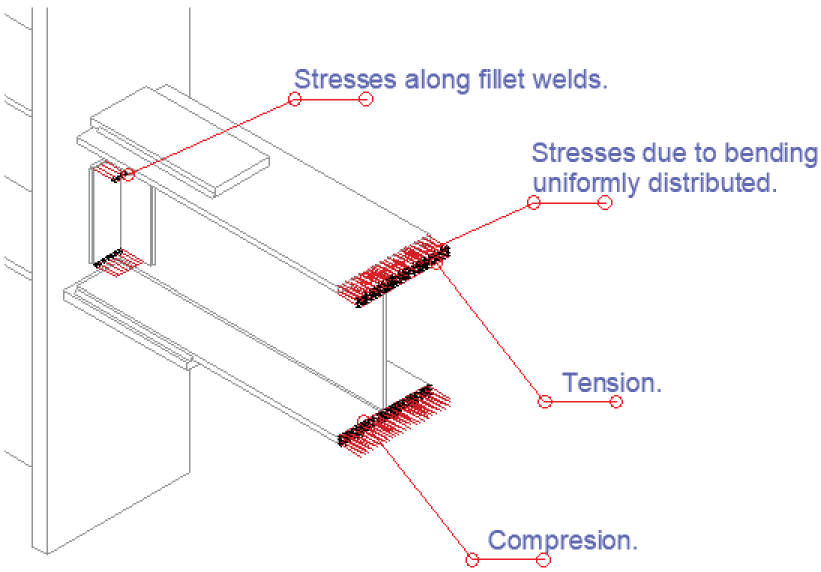


Figure 48: Criteria employed for producing bending moment in configuration with angles seated (*represented by vectors with tangential/normal direction over some surface of the elements to join.*)

- Shear loads will be located after the free-edge of bending plates.
- Column will no be affected for any kind of mechanical action.
- Shear and the normal forces will be uniformly distributed over the surface of cross-section, for both configurations, the loads will be located at the same points/areas (Figure 47) to get congruent comparisons.

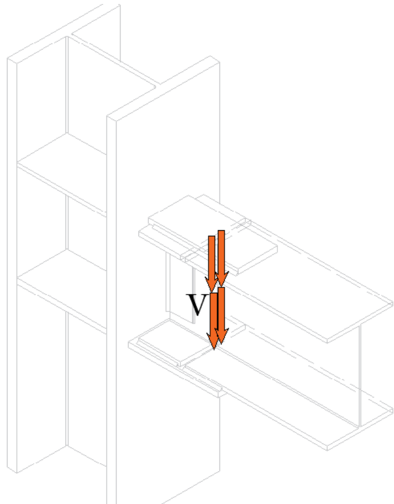


Figure 49: Punctual loads applied after the edge of fillet weld located at beam web and shear plates/angles set.

- The software has already integrated the algorithms that either will administer or assign *"an equivalent portion of the surface load configured turned into a punctual load to be later transfered over the nodes of elements given from the meshing proceeding"*.
- The discretization is based on a *structured* mesh with tetrahedral elements for the couple of test models. The non-qualified prototype has very refined details over the domain, complicating relatively the meshing process.
- The boundary conditions (restrictions) will be imposed to the surfaces represented in cian color as the Figure 48 shows, and consequently, an algorithm from the program will declare the points belonging to the surfaces imposed with a restriction on displacement. For the column will be in the x, y *and* z directions, and in the beam cross-section surface just for x in a way of avoiding torsion and recreate continuity for ductile steel frames.

About dimensions and another premises:

- Symmetry is assumed in where beams with joint elements are placed, these also perfectly symmetric respect to W profile geometrical properties.
- There is continuity on the planes where bending plates and the flange of column, this, to observe the resultant stresses needed for designing the groove weld (Complete/partial Joint Penetration), and the most appropriate process of welding and features.
- The "leg size" for fillet welds will fit inside the width of the beam or plates used, also, the beam with plates and profile represented for column are perfectly centered.
- There is total continuity between the surfaces that join beam to the leg of angles or shear plates. Besides, this condition also accomplishes for the bending plates connected to the flange of beams; the stiffening plates also maintains the continuity according the drawing and trace of the model created virtually.

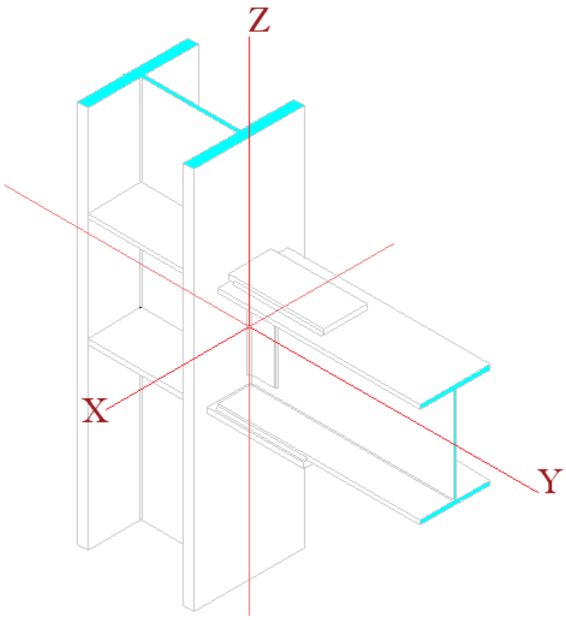


Figure 50: Boundary conditions applied to the surfaces of approached problems.

- The beam, for both steel connections, will have one centimeter offset (or gap) from flange of column (constant for all models recreated).
- There is a little offset of 3mm ($\frac{1}{8}in$) between the beam and the bending plates (constant for all models recreated).
- The leg of angle welded to the flange of column will be seated with such dimension that allows to fit inside the leg of flange of the beam used, and its corresponding bending plate. This, due to the order in how dimensions of elements were initially programmed in problem types.
- The offset between shear plates or the leg welded to the flange of column for unconventional model is considered as constant of 2mm.
- The solver is a tool generated by researchers from University of Guanajuato (PhD. Salvador Botello, and Ms.C. Humberto Esqueda Oliva), named "**MEFI**". The solver affords solving problem for plane stress, plain strain and three-dimensional problems under elastic regime.
- The residual stresses will be neglected (*or not modelled* for finite element analysis).
- There is a pure continuity between the localized surfaces of the welds with the same plane of the element which is being joined.
- The height of the column for every connection tested is $200cm$ ($6ft$ *and* $8in$).
- The length of the segments for beams is $60cm$ ($2ft$).
- The problem type code models just one W-shape to be connected and used for beam/column.

About VBA Excel Spreadsheets:

- The functions and mathematical expressions for values of strength consider effects of residual stresses and a deep background founded on experimental studies.

- The bending moment is decomposed as figure 20 shows, and rigid plates must be designed to resist uniformly the pair of forces. The methodology for this tool does not display results for effects of shear adjusted to the flexural moment imposed.

- The spreadsheet does not display any information about stresses distribution, strains, displacements, or something alike.

- The weld lines illustrated on Figure 49 will define how the weld lines join the flange of column to angles, besides, this diagram represents how the dimensions are distributed to analyse strength.
  The distance "2a" corresponds to 60-65% of the total length of the angle depth considered to resist shear (the depth of the angle is the sum of $2a + 2b$). The weld figures F-1 and F-2, with lengths "b" and "c" (red lines) are supposed to be resilient enough to support the plastic moment equivalent to the plastic modulus of the web for the beam. The coordinate $0, 0$ is the centroid of W beam section perfectly aligned with the couple of angles to be connected.

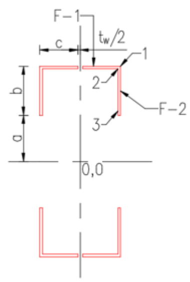


Figure 51: Set of welds over flange of column for models to validate.

- The strength of stiffening plates will be designed according to the use of 2 pairs of prismatic plates with the same thickness of the bending plates. The width expands from free-edge of flange to the web of the column. This, because dimensional features were assumed and programmed in the problem type code to maintain continuity and symmetry along the beam (dimensions could be smaller but remain according to last to match the congruence between the couple of solutions).

- There will be also checked the limit states for *Flange Local Bending, Web Local Yielding, and Web Local Crippling*. This particularly for purposes merely informative, and to be considered for design of stiffening plates reinforcement, and following the procedure as ANSI/AISC-360-16 dictates (*Chapter J.10: Flanges and Webs with Concentrated Forces*) for the VBA Excel spreadsheets.

According to last, it is important to follow the mentioned geometrical restrictions for both configurations; this, because the programming code has strictly defined the order of geometric parameters as a fixed and logical condition. So that, any anomaly detected or any incompatible consideration will be enough to guarantee failure in creating the geometry.
The concepts from subsection 3.5 will govern the calculating of solution for qualified models tested (stiffened and unstiffened), the load will be distributed uniformly over the bending plates surface to recreate conditions of a moment acting. The shear force will be distributed over the shear plates surfaces. For the premises mentioned in the subsection, the 100% of the forces that produce bending moment will be taken for the rigid plates.

For each geometry to recreate, there will be also drawn and compared a twin model destined to be studied for every designation of steels: ASTM A-36, ASTM A572 Gr. 50. The steel characteristics/designation for beams and columns will remain constant for qualified/non-qualified prototypes tested (ASTM A572 Gr. 50, $F_y = 3515 kg/cm^2$ $F_u = 4570 kg/cm^2$).
The design forces for the first two models arise from structural analysis on software *SAP2000*, the

models 0 and 3 will consider an 80% of nominal yield moment and a shear of 45% of nominal shear strength (according to W-shape selected for the beam). The service loads will be calculated as the ultimate forces divided by 1.40.
Observation: These premises will stand fixed and respected either for angles-coupled or qualified configurations.

Limit States from Chapter J.10: Flanges and Webs with Concentrated Forces

Flange Local Bending.
This section applies to tensile single-concentrated forces and the tensile component of double-concentrated forces [10].
When the concentrated force to be resisted is applied at a distance from the member end that is less than $10t_f$, R_n shall be reduced by 50%.

$$R_n = 6.25F_{yf} \cdot t_f^2 \tag{105}$$

Where:

- F_{yf} = yield strength of column flange.
- t_f = thickness of column flange.
- $\phi = 0.90$ (LRFD), $\Omega = 1.67$ (ASD).

If the length of loading across the member flange is less than $0.15b_f$, where b_f is the member flange width, Equation 105 need not be checked.
When the concentrated force to be resisted is applied at a distance from the member end that is less than $10t_f$, R_n shall be reduced by 50%. When required, a pair of transverse stiffeners shall be provided.

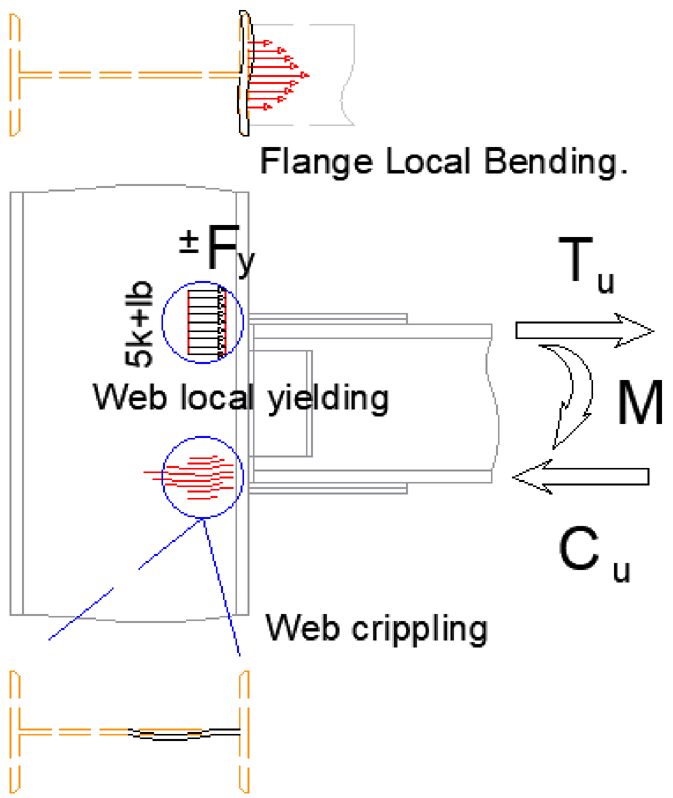


Figure 52: Local failures over the zone connected between column and beam (dark contour indicates the deformed shape of failure).

Web Local Yielding.

This section applies to single-concentrated forces and both components of double concentrated forces. C_u is a compression force transferred to the column [10].

$$R_n = F_{yw} \cdot t_w \ (5k + l_b) \tag{106}$$

- F_{yw} = yield strength of column web.
- t_w = thickness of web.
- k = eccentricity of W shape (distance from outer face of the flange to the web toe of the fillet).
- l_b = length of bearing (not less than k for end beam reactions).
- $\phi = 1.00$ (LRFD), $\Omega = 1.50$ (ASD).

Web Local Crippling.

This section applies to compressive single-concentrated forces or the compressive component of double-concentrated forces.

$$R_n = 0.80 t_w^2 \left[1 + 3 \left(\frac{l_b}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{E \cdot F_{yw} \cdot t_{fc}}{t_w}} Q_f \tag{107}$$

- $d = h$, full nominal depth of member.
- $Q_f = 1.0$ for wide-flange sections.
- $\phi = 0.75$ (LRFD), $\Omega = 2.00$ (ASD).

Design of stiffner-to-column plates.

The thickness of continuity plates t_s will be taken equal to the magnitude corresponding to the thickness for rigid bending plates (upper and lower), and also considered as perfectly aligned with them. The VBA spreadsheet makes the compute for 3 types of stiffner plates: optimal (required), mid plate (from web outer surface to the mid distance of free edge of flange), and the full plate (from web outer surface to the free edge of flange). The effective width b_s is considered as Figure 54 pictures, notice that revisions must realize of area reduction by the clips (small cuts at the vertexes to assembly in the interior of W shape).
The width will be calculated considering the thickness continuity, using the yield stress for each category (ASTM A36, and later ASTM A572 Gr. 50). Plates will be designed as short columns that reach yield stress according to next expression

$$b_s = \frac{C_u - min(R_{FLB}, R_{WLY})}{(\phi \ or \ \Omega) F_y \cdot t_s} \tag{108}$$

where

- $C_u \ or \ T_u$ = design force (M_u divided by beam depth h_b).
- R_{FLB} = flange local bending strength.
- R_{WLY} = web local yielding strength.
- F_y = yield strength of union plates (same for shear and bending according to this thesis).
- t_s = thickness of stiffner plate (it might take another value for another purposes).
- $\phi = 0.90$ (LRFD), $\Omega = 1.67$ (ASD).

The difference in numerator is absolute (positive quantity).

At last, dimensions of such plates must accomplish some dimensional requirements (enlisted below):

$$b_s + \frac{t_{wb}}{2} \geq b_{fb} \ (I)$$

$$t_s \geq t_{fb} \ (II)$$

$$\frac{b_s}{t_s} < 0.56\sqrt{\frac{E}{F_y}} \ (III)$$

where:

- t_s = reinforcement plate thickness.
- b_s = width of project assigned for continuity plates.
- t_{wb} = web thickness of beam.
- t_{fb} = flange thickness of beam.
- b_{fb} = flange width of beam.
- F_y = yield strength of rigid plate.

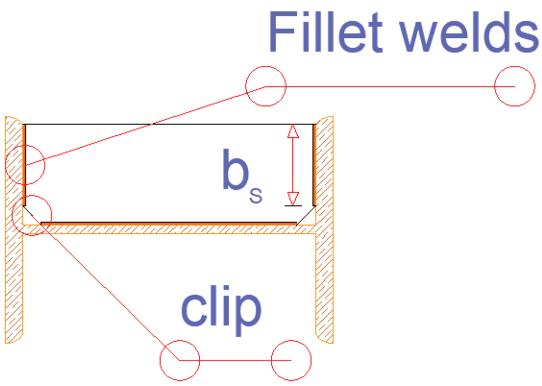


Figure 53: Features for stiffner-to-column for column).

The fillet welds employed to hold the continuity plates will be analysed according to premises of previous chapter (section 3.4). Such joints must provide resistance enough to support the design load C_u, the fillet welds will be set over the contour lines with the clips done. The clips cut from the plate must have a 1:1 relationship (base and height scalars for triangle). A pair of lines will be welded at upper and lower edges.
The VBA spreadsheet will command the compute the design parameter for each configuration of rigid plate. Notice that if table displays $A_{req} = 0$ for optimal plate, the meaning depicts that reinforcement is not required according to calculations.

5.3.3 Qualified Bending Moment Steel Connections (Unstiffened and Stiffened)

The models will be tested considering resistance with and without stiffening plates along the axis of bending plates with the same thickness. The stiffening plates will be considered attached to the connection by fillet welds (not drawn for pre process). The shear plate to make the joint of the beam-column has a 2mm offset from the column flange and the edge of the plate, and there is also an offset of 3mm between bending plates and the beam.

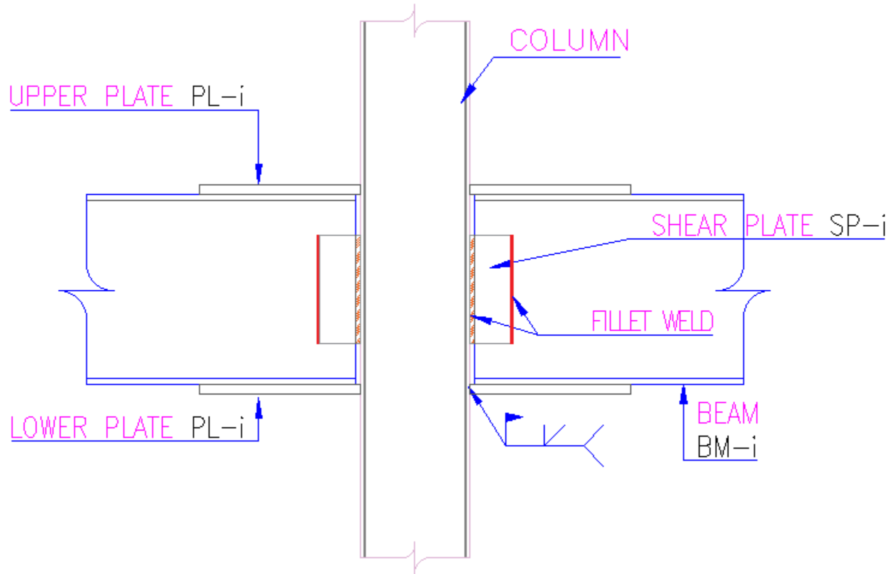


Figure 54: Features and parameters established for models to study (CUT -XX-FI).

As the next picture shows, the beam is considered that fits in the lower bending plate, joined by fillet and penetration welds, and that the flange of column is wider than the flange of the beam. For every model studied and to be calculated will have an offset between beam and column of 1cm by default.

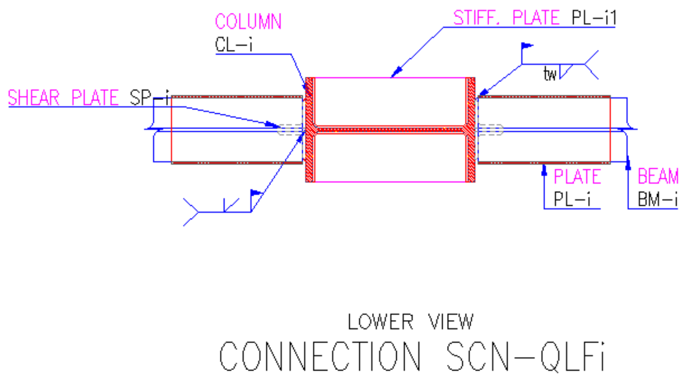


Figure 55: Features and parameters established for models to study (LOWER VIEW-FII).

In the other hand, the upper view displays how the beam is considered that fits in the upper bending plate, joined by fillet and penetration welds, and that the flange of column is wider than the flange of the beam. For the upper plate, the parametric geometries of the Problemtypes consider that the width of the upper plate plus the sum of twice times the leg size of weld (w).

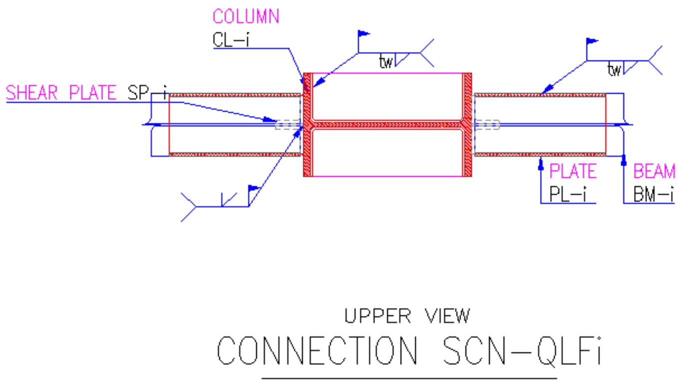


Figure 56: Features and parameters established for models to study (UPPER VIEW-FIII).

The models proposed there going to be named as the sequences of elements are proposed as follows:

MODEL	**Beam (BM)**	**Column (CL)**
M0-BM0-CL0-STF/UNS	W12x30	W16x100
M1-BM1-CL1-STF/UNS	W16x45	W16x100
M2-BM2-CL2-STF/UNS	W18x46	W16x100
M3-BM3-CL3-STF/UNS	W24x55	W16x100

The beam properties for the connections have the next geometrical dimensions:

Measure (cm)	(BM_0)	(BM_1)	(BM_2)	(BM_3)
h	31.24	40.89	45.97	59.94
b_f	16.56	17.88	15.39	17.81
t_f	1.12	1.44	1.54	1.28
t_w	0.66	0.88	0.91	1.00

The column properties for the connections have the next geometrical dimensions:

Measure (cm)	(CL_0)	(CL_1)	(CL_2)	(CL_3)
h	43.18	43.18	43.18	43.18
b_f	26.42	26.42	26.42	26.42
t_f	2.50	2.50	2.50	2.50
t_w	1.49	1.49	1.49	1.49

5.3.4 Optimized Configuration for Bending Moment Steel Connections (Unstiffened and Stiffened)

The models will be tested considering resistance with and without stiffening plates along the axis of bending plates with the same thickness. The stiffening plates will be considered attached by fillet welds. The angle used to make the joint of the beam-column has a 2mm offset from column flange to the leg of angles couple, and there is also an offset of 3mm between bending plates and the beam.

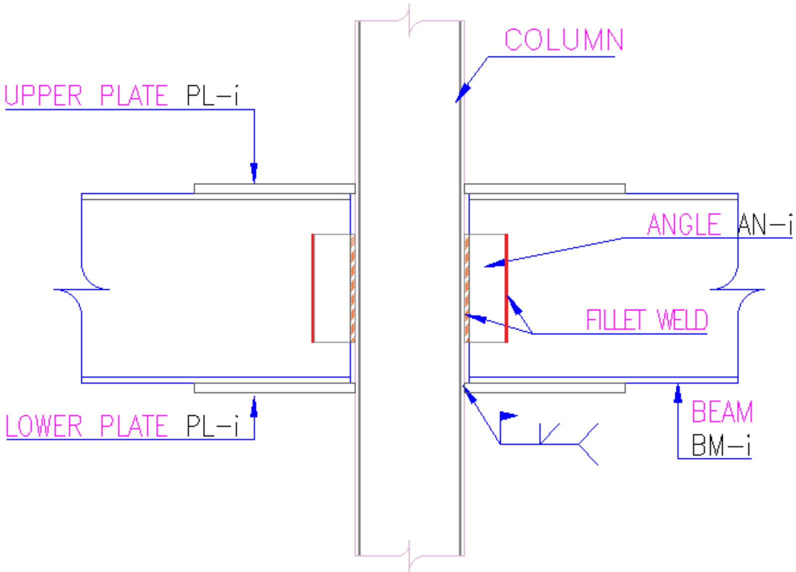


Figure 57: Features and parameters established for models to study (CUT -XX-FI).

As the next picture shows, the beam is considered that fits in the lower bending plate, joined by fillet and penetration welds, and that the flange of column is wider than the flange of the beam. For every model studied and to be calculated will have an offset between beam and column of 1cm by default.

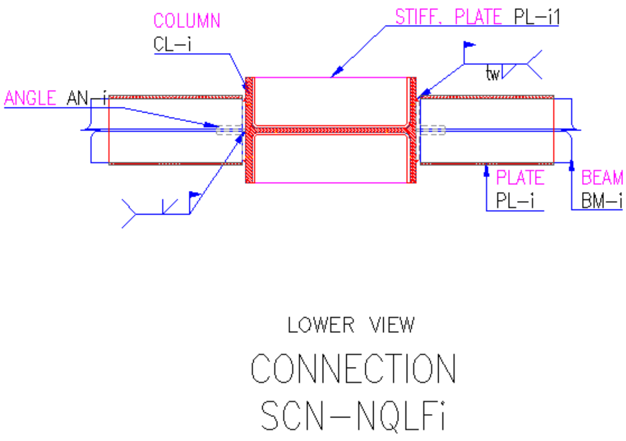


Figure 58: Features and parameters established for models to study (LOWER VIEW-FII).

In the other hand, the upper view displays how the beam is considered that fits in the upper bending plate, joined by fillet and penetration welds, and that the flange of column is wider than the flange of the beam. For the upper plate, the parametric geometries of the Problemtypes consider that the width of the upper plate plus the sum of twice times the leg size of weld (w).

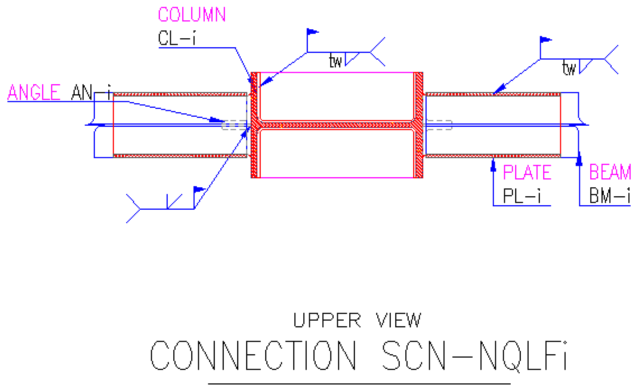


Figure 59: Features and parameters established for models to study (UPPER VIEW-FIII).

The models proposed there going to be named as the sequences of elements are proposed as follows:

MODEL	**Beam (BM)**	**Column (CL)**
M0-BM0-CL0-STF/UNS	W12x30	W16x100
M1-BM1-CL1-STF/UNS	W16x45	W16x100
M2-BM2-CL2-STF/UNS	W18x46	W16x100
M3-BM3-CL3-STF/UNS	W24x55	W16x100

The beam properties for the connections have the next geometrical dimensions:

Measure (cm)	(BM_0)	(BM_1)	(BM_2)	(BM_3)
h	31.24	40.89	45.97	59.94
b_f	16.56	17.88	15.39	17.81
t_f	1.12	1.44	1.54	1.28
t_w	0.66	0.88	0.91	1.00

The column properties for the connections have the next geometrical dimensions:

Measure (cm)	(CL_0)	(CL_1)	(CL_2)	(CL_3)
h	43.18	43.18	43.18	43.18
b_f	26.42	26.42	26.42	26.42
t_f	2.50	2.50	2.50	2.50
t_w	1.49	1.49	1.49	1.49

6 Results: Union Plates ASTM A36

6.1 Qualified Bending Moment Configurations

6.1.1 Model 0.

Input Data

Name of model: M0-BM0-CL0-A36, (arbitrary model)
Beam: (BM_0) W12x30 — Column: (CL_0) W16x100 — Steel Designation: ASTM-A572 Gr. 50
$F_y = 3515 kg/cm^2$ $(50ksi)$ — $F_u = 4570 kg/cm^2$ $(65ksi)$ — Thickness governant: Column
The flange of beam fits in the flange of column $(0.85 b_{fc} > b_{fb})$.
From beam:

- M_{pf}: $1.99\ e+06\ kg \cdot cm$ flanges plastic moment contribution.
- M_{pw}: $0.49\ e+06\ kg \cdot cm$ web plastic moment contribution.
- M_p: $2.48\ e+06\ kg \cdot cm$ total plastic moment.
- M_y: $2.22\ e+06\ kg \cdot cm$ yield moment.
- $\dfrac{M_{pf}}{M_p} = 80.35\%$ percentage contribution of flanges respect to total plastic moment.
- $\dfrac{M_{pw}}{M_p} = 19.65\%$ percentage contribution of web respect to total plastic moment.

Design forces:

- $V_u = 1.29\ e+04\ kg$ ultimate shear load.
- $V_s = 0.92\ e+04\ kg$ service shear load.
- $M_u = 1.78\ e+06\ kg \cdot cm$ ultimate moment.
- $M_s = 1.27\ e+06\ kg \cdot cm$ service moment.

Rigid Upper Plate

From beam:

Flexural strength.		
$F(kg \cdot cm)$	LRFD (ϕM_n)	ASD $(\frac{M_n}{\Omega})$
Nominal moment	2.23e+06	1.49e+06

$F_{ut} = F_{uc} = 0.80\ e+05\ kg$ ultimate tension/compression force to design bending plates (according to M_u).
$F_{st} = F_{sc} = 0.41\ e+05\ kg$ service tension/compression force to design bending plates (according to M_s).

Resultant dimensions:

- $b_p = 14.29cm$ $(5.625in)$ width of plate.
- $t_p = 3.18cm$ $(1.25in)$ thickness of plate.
- $L_p = 30.00cm$ $(12.00in)$ length of plate.
- $\psi = 1.12$ shape factor.

Upper plate nominal force.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal yield force	1.03e+05	0.69e+05
Nominal rupture force	1.39e+05	0.93e+05

Fillet Welds for Rigid Plates

Results:

- $EXX/FX = 4900kg/cm^2$ $(70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.80cm$ minimum leg size of weld.
- $tw_{max} = 2.98cm$ maximum leg size of weld.
- $w_p = 1.00cm$ leg size of weld for design of project.
- $Lw_{req} = 51.01cm$ length of weld required.
- $Lw_p = 58.00cm$ length of weld to be adjusted to the project.
- $Lw_{eff} = 58.00cm$ effective length of proportioned weld.
- $R_{nw} = 2078.58kg/cm$ theoretical strength of weld.

Nominal strength of fillet weld.		
$F(kg/cm)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal strength	1558.94	1039.29

Nominal force due to fillet welds.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal force	0.90e+05	0.60e+05

Rigid Lower Plate

Results:

- $b_p = 21.59cm$ $(8.50in)$ width of plate.
- $t_p = 1.91cm$ $(0.75in)$ thickness of plate.
- $L_p = 30.00cm$ $(12.00in)$ length of plate.
- $r = 0.55cm$ gyration radius of plate.
- $k = 0.65$ effective length factor.
- $kL_p = 19.50cm$ effective length.
- $\frac{kL_p}{r} = 35.46$ slenderness ratio (adimensional).
- $F_e = 16.01\ e+03\ kg/cm^2$ Euler stress.
- $F_{cr} = 2.37\ e+03\ kg/cm^2$ critical stress.
- $P_n = 0.97\ e+05\ kg$ theoretical compression force resisted.

Compression force resisted by lower plate.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal force	0.88e+05	0.58e+05

Shear Design

Beam checking:

- $\frac{h}{t_w} = 47.31$ slenderness ratio of web.
- $k_v = 5.34$ (a=0, no transverse stiffeners) adimensional value.
- $C_{v1} = 1.00$ adimensional value.
- $A_w = 20.63cm^2$ area of the web.

Shear strength of beam section.		
$F(kg)$	LRFD (ϕV_n)	ASD ($\frac{V_n}{\Omega}$)
Nominal shear resistance	3.92e+04	2.61e+04

Shear plates:

- $b_p = 10.16cm$ $(4.00in)$ width of plate.
- $t_p = 0.95cm$ $(0.375in)$ thickness of plate.
- L_p *or* $h = 15.24cm$ $(6.00in)$ length of plate.

Strength of shear plates (the pair).		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal yield strength	3.97e+04	2.64e+04
Nominal rupture strength	5.33e+04	3.55e+04

Results for fillet welds of shear plates:

- $EXX/FX = 4900kg/cm^2$ $(70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.50cm$ minimum leg size of weld.
- $tw_{max} = 0.75cm$ maximum leg size of weld.
- $w_p = 0.50cm$ leg size of weld for design of project.
- $Lw_p = 30.48cm$ length of weld proportioned.
- $Lw_{eff} = 30.48cm$ effective length of proportioned weld.

Nominal strength of fillet welds for shear plates.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal strength	2.38e+04	1.58e+04

Additional quantities.		
Weight	*Length of fillet weld (total)*	*Length of groove weld (total)*
25.03 Kg	30.48 cm of 5 mm, and 116.00 cm of 10 mm	35.88 cm

Flanges and Webs with concentrated forces (Chapter J.10)

Flange local bending.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	2.47e+04	1.64e+04
Column	12.37e+04	8.23e+04

From column:

- t_w = 1.49cm thickness of web.
- k = 3.53cm eccentricity of W shape.
- l_b = 3.91cm bearing length (min(t_p,t_{fb})+2w); where:
  - w is the leg size of weld project (w_p).
  - t_p is the thicker thickness of the bending plates.
  - t_{fb} is the thickness of the flange beam.

Web local yielding.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	10.74e+04	7.16e+04

From column:

- t_f = 2.50cm thickness of flange.
- h = 43.18cm depth of W shape.
- l_b = 3.91cm bearing length.
- Q_f = 1.00 adimensional factor.

Web local crippling.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	16.36e+04	10.91e+04

Transverse stiffeners (Chapter G2, Section 3 from ANSI/AISC-360-16):

Feature	**Beam**	**Column**
h	31.24cm	43.18cm
t_w	0.66cm	1.49cm
$\frac{h}{t_w}$	47.31	29.06
$2.46\sqrt{\frac{E}{F_{yw}}}$	59.26	59.26
Requirement?	NO	NO

- F_{yw} = 3515kg/cm^2 yield stress.
- Transverse stiffeners **not necessary if**:

$$\frac{h}{t_w} < 2.46\sqrt{\frac{E}{F_{yw}}}$$

Stiffening/Continuity Plates

- $C_u = T_u$: 0.00
- A_{req}: 0.00
- $\frac{b_{fb}}{3}$: $5.52cm$
- $0.56\sqrt{\frac{E}{F_y}}$: 15.90

For upper plate ($t_{up} = 3.18cm$ $(1.25in)$):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	33.53	34.37	10.56	7.63e+04	5.08e+04
Mid plate	16.76	34.37	5.28	3.82e+04	2.54e+04
Optimal	0.00	34.37	0.00	0.00	0.00

For lower plate ($t_{lp} = 1.91cm$ $(0.75in)$):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	20.12	34.37	10.56	4.58e+04	3.05e+04
Mid plate	10.06	34.37	5.28	2.29e+04	1.52e+04
Optimal	0.00	34.37	0.00	0.00	0.00

Note: strengths (kg) are expressed for just one plate.
For welds (two alternatives, it considers upper and lower edges):

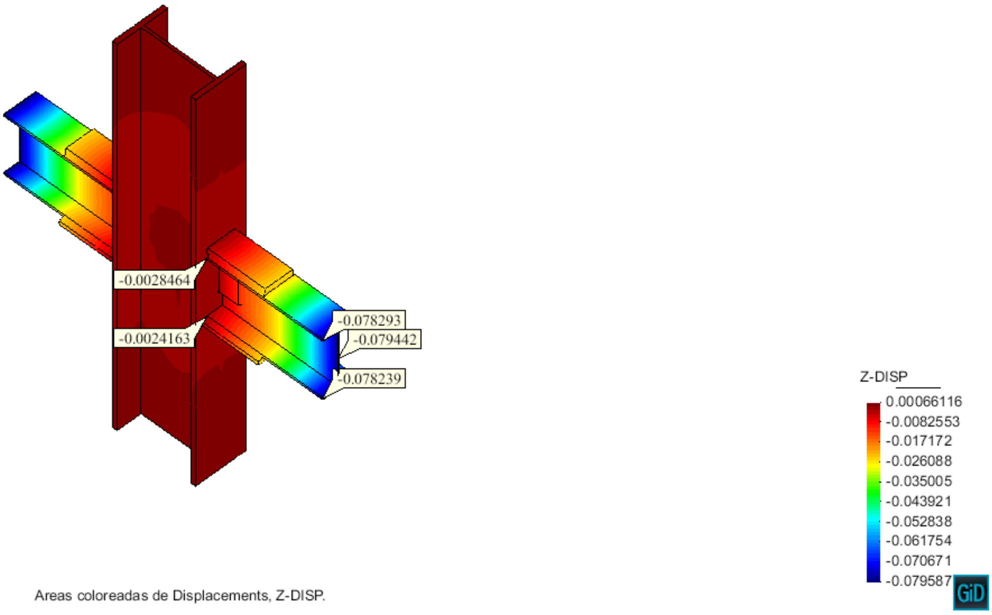
- t_{w1} (weld leg size for alternative 1) = $0.30cm$ $(\frac{1}{8})''$
- t_{w2} (weld leg size for alternative 2) = $0.50cm$

Plate	$LRFD(\phi R_{n1})$	$ASD(\frac{R_{n1}}{\Omega})$	$LRFD(\phi R_{n2})$	$ASD(\frac{R_{n2}}{\Omega})$
FP/MP/OP	3.21e+04	2.14e+04	5.36e+04	3.57e+04

Observations

- Plates have a work performance up to 55% for yield (M_u), and about of 80-85% for ultimate design forces.
- Strength is delimited by resistance of welds.
- There is not need of stiffener elements.

Finite Element Analysis results: Model 0, Displacements.



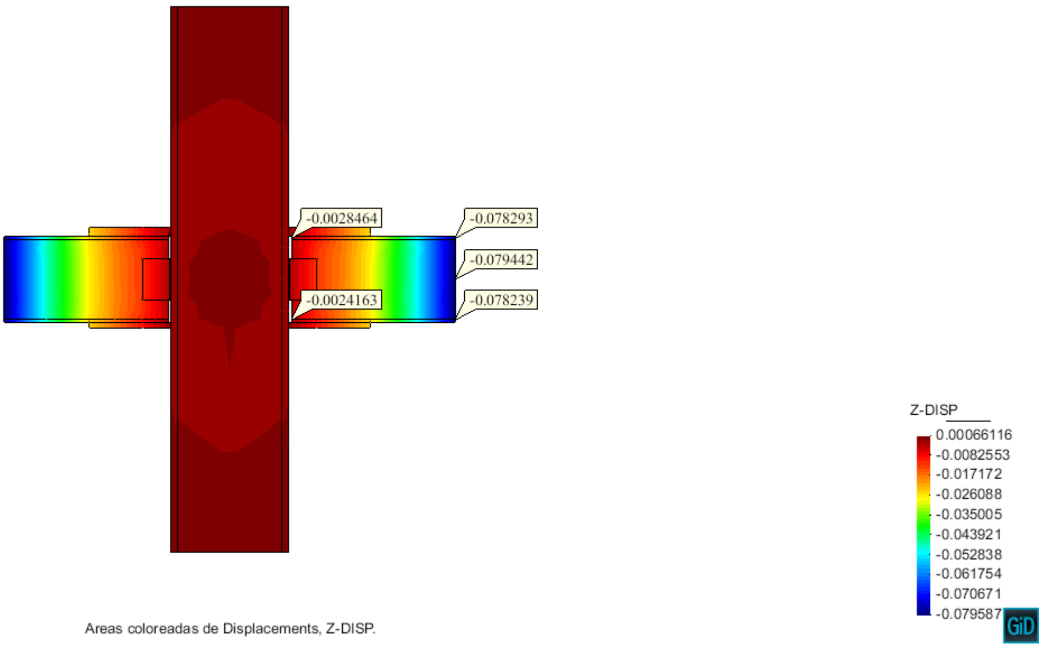


Figure 60: Vertical displacements (Z-axis) in M0-A36 conventional model.

Observations:

- The upper edges rotate and deflect more than the lower ones.

Finite Element Analysis results: Model 0, Tension stresses S_{yy}.

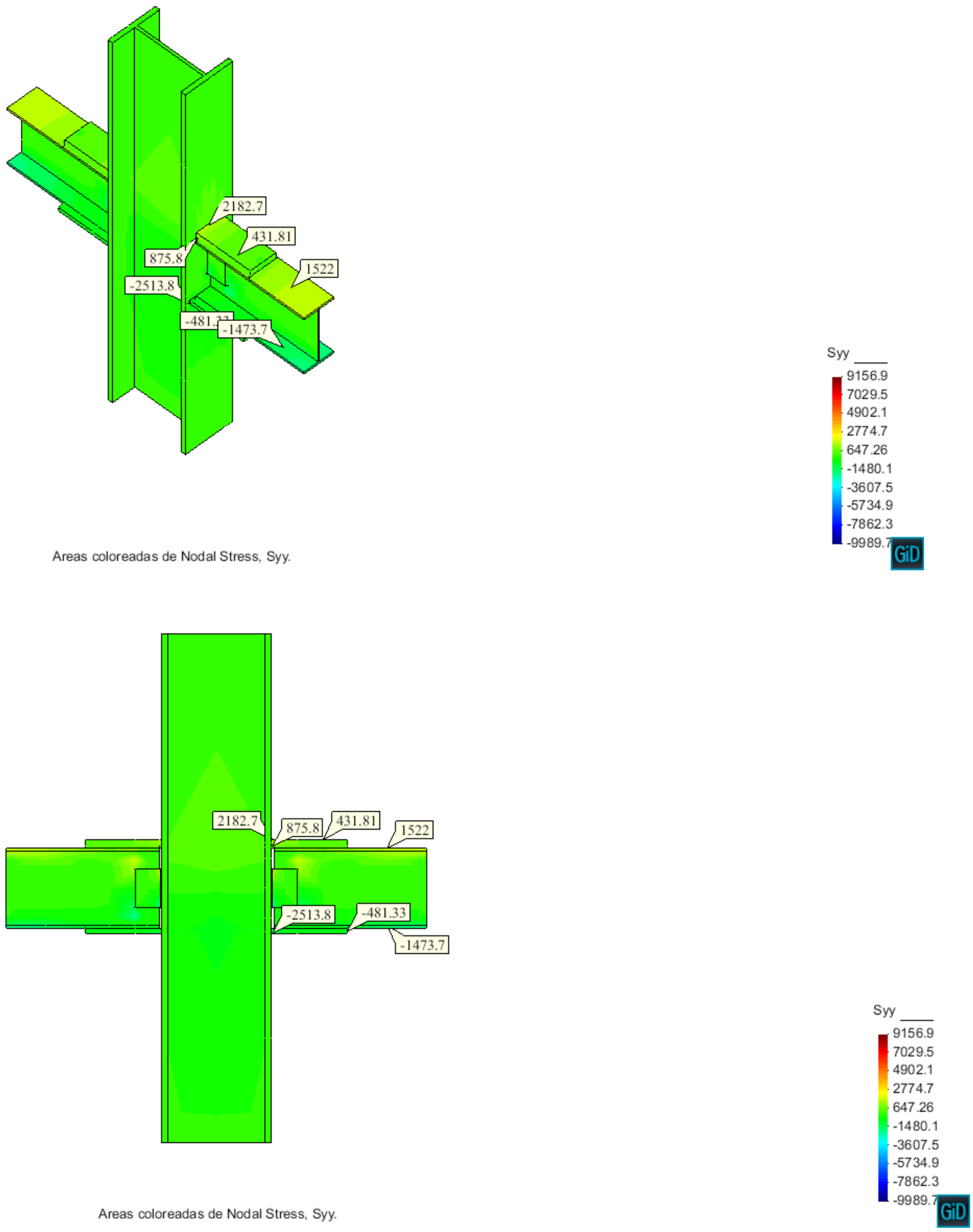
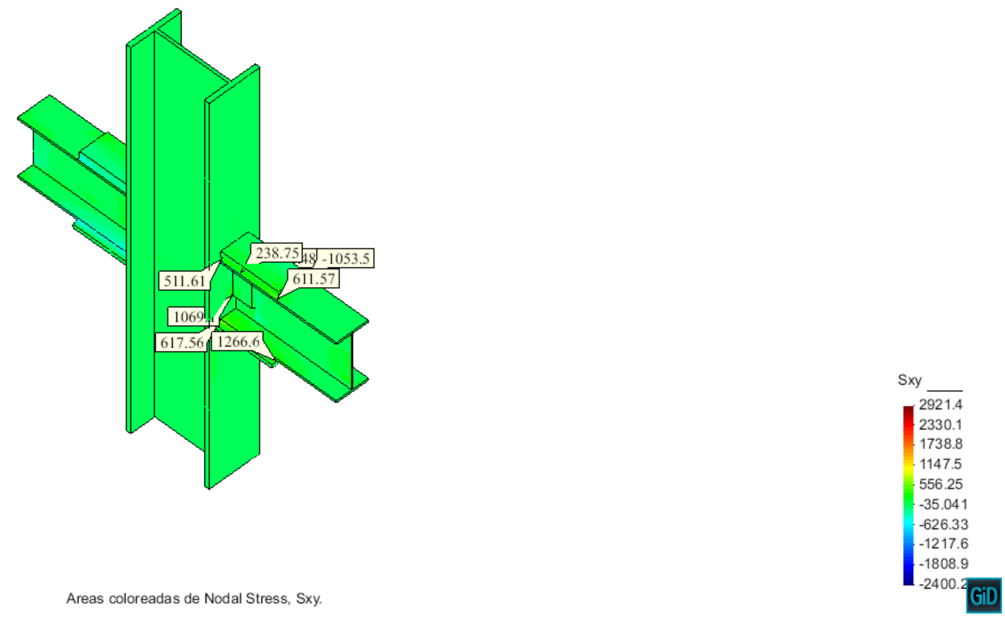


Figure 61: Axial stresses (Y-axis, SYY) in M0-A36 conventional model.

Observations:

- Both plates work efficiently under yield stress.
- Tensile stress over the upper plate reduces as the position gets far from the groove weld location.

Finite Element Analysis results: Model 0, Shear stresses S_{xy}.



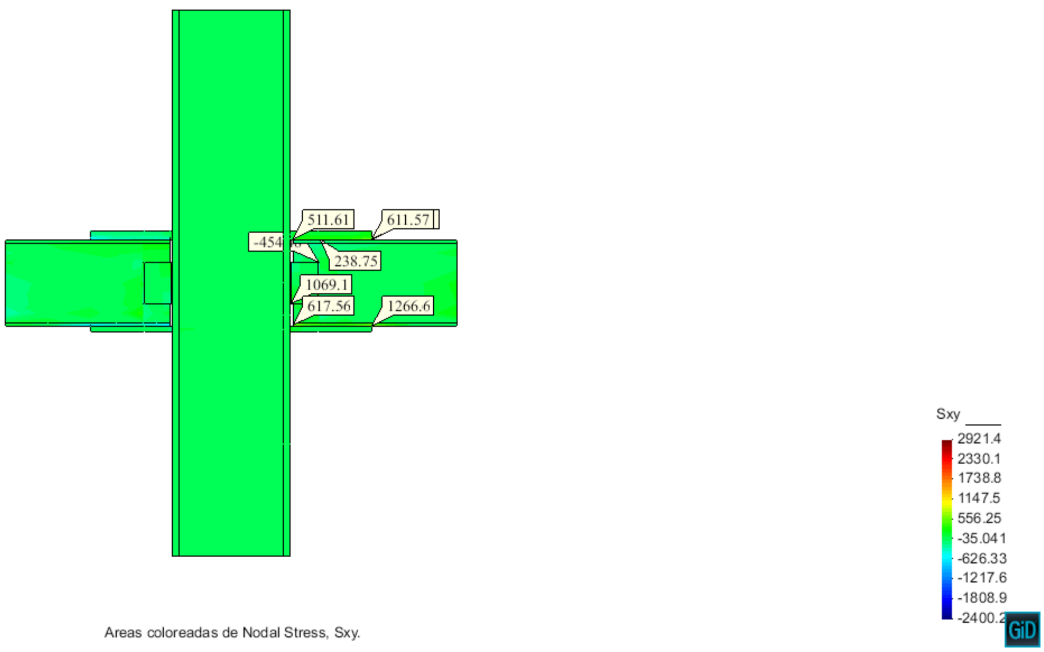


Figure 62: Shear stresses (XY plane, SXY) in M0-A36 conventional model.

Observations:

- Shear stresses along fillet welds works under their theoretical and nominal strength.

Finite Element Analysis results: Model 0, Shear stresses S_{yz}.

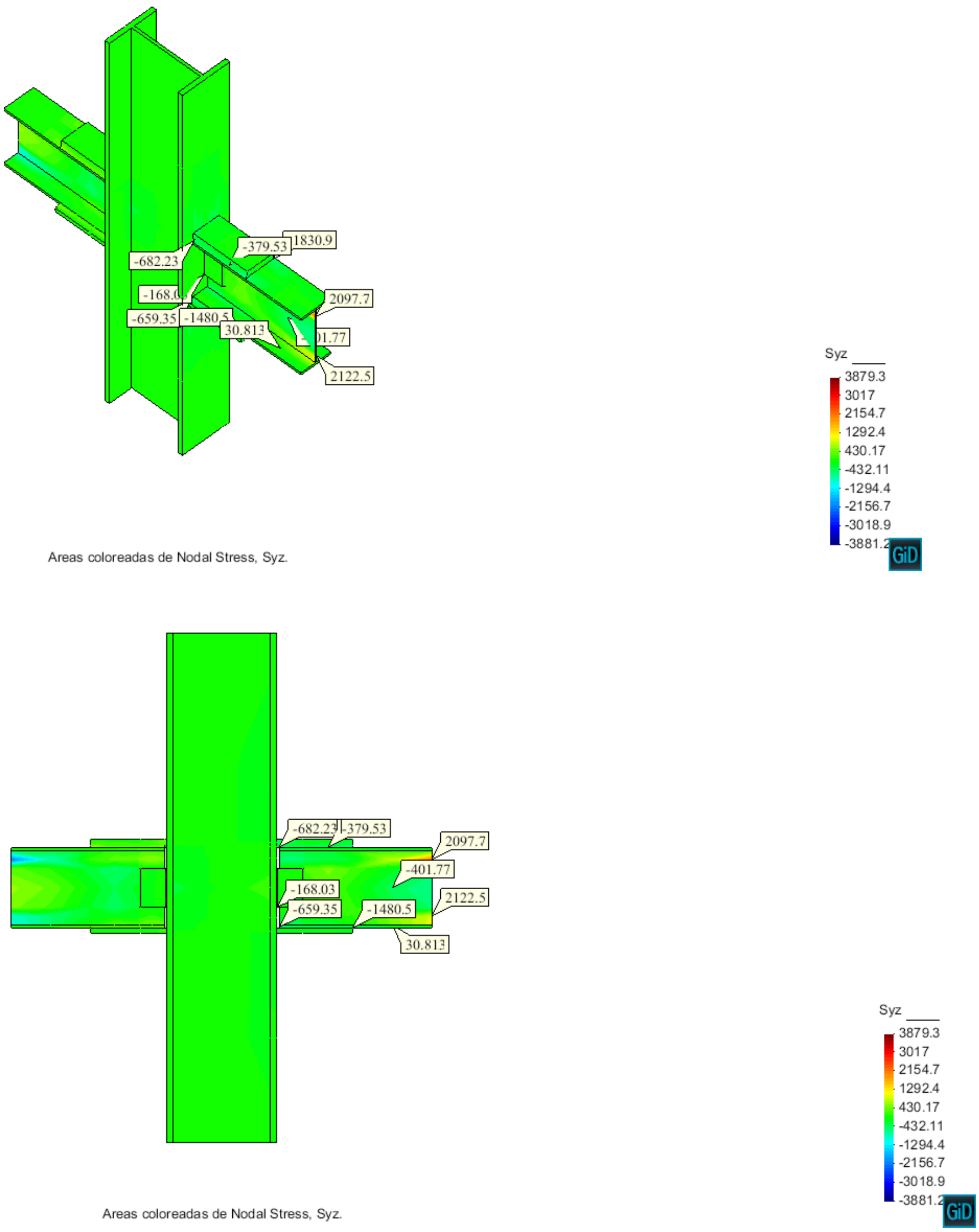
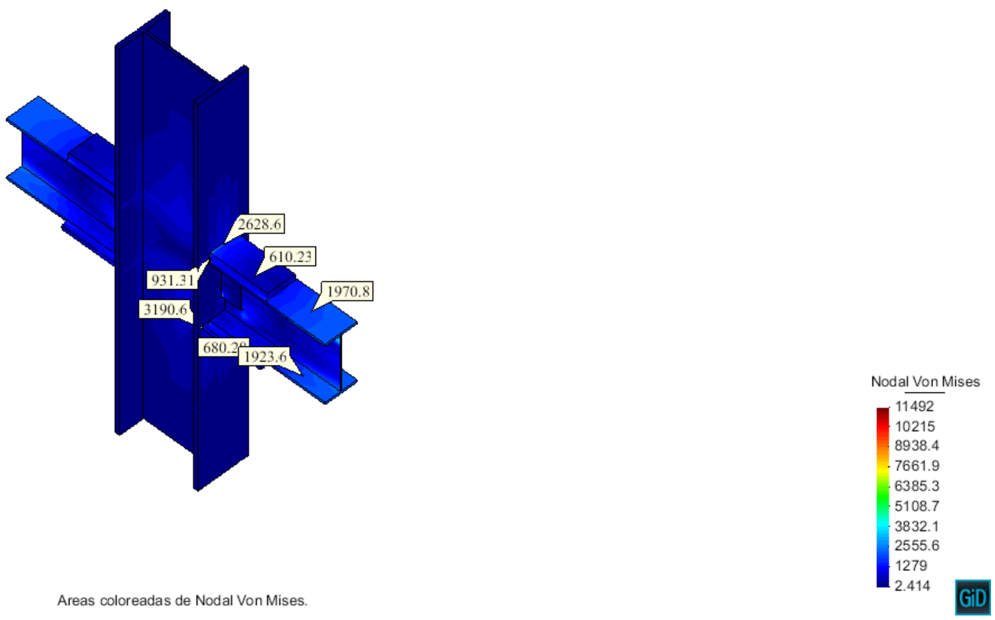


Figure 63: Shear stresses (YZ plane, SYZ) in M0-A36 conventional model.

Observations:

- The most of shear stresses along fillet welds works under their theoretical and nominal strength except those regions betweens the small gaps and joined plates/elements.
- There is a high concentration of shear stresses between the web and flanges for the W shape.

Finite Element Analysis results: Model 0, Von Mises stresses VM_s.



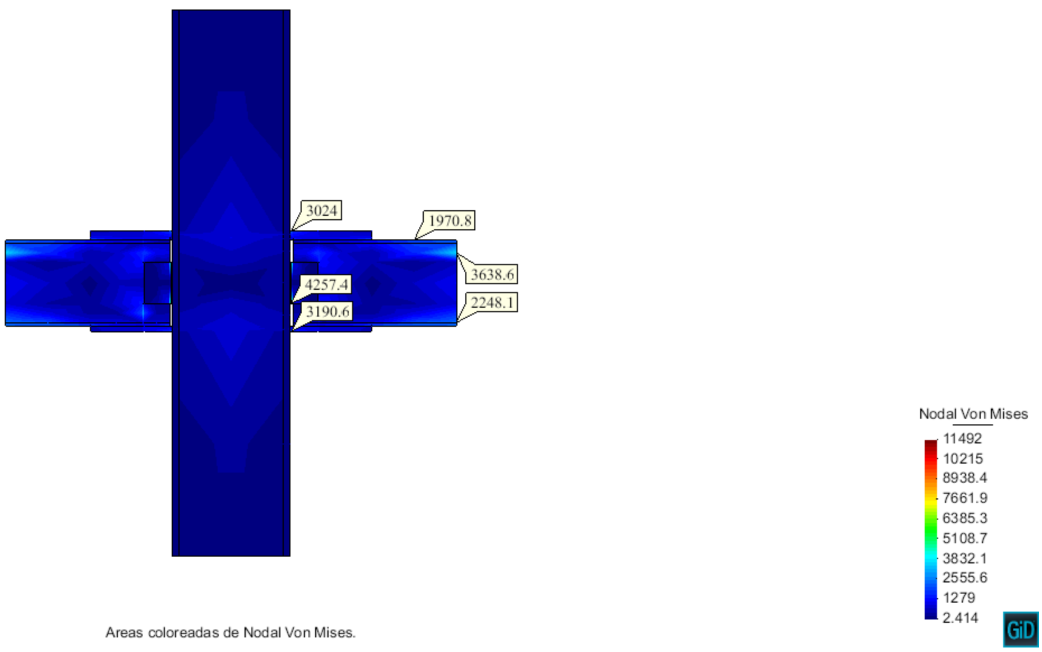


Figure 64: Von Mises stresses in M0-A36 conventional model.

Observations:

- The yield surfaces arises from the bending plates connected to the column flange, the small gaps between welded elements and transitions among web and flanges. Even so, the stresses are under ultimate tensile stress.

6.1.2 Model 1.

Input Data

Name of model: M1-BM1-CL1-A36, (model for offices building)
Beam: (BM_1) W16x45 — Column: (CL_1) W16x100 — Steel Designation: ASTM-A572 Gr. 50
F_y= $3515kg/cm^2$ $(50ksi)$ — F_u= $4570kg/cm^2$ $(65ksi)$ — Thickness governant: Column
The flange of beam fits in the flange of column $(0.85b_{fc} > b_{fb})$.
From beam:

- M_{pf}: $3.62\ e+06\ kg \cdot cm$ flanges plastic moment contribution.
- M_{pw}: $1.11\ e+06\ kg \cdot cm$ web plastic moment contribution.
- M_p: $4.74\ e+06\ kg \cdot cm$ total plastic moment.
- M_y: $4.19\ e+06\ kg \cdot cm$ yield moment.
- $\frac{M_{pf}}{M_p} = 76.50\%$ percentage contribution of flanges respect to total plastic moment.
- $\frac{M_{pw}}{M_p} = 23.50\%$ percentage contribution of web respect to total plastic moment.

Design forces:

- $V_u = 3.01\ e+04\ kg$ ultimate shear load.
- $V_s = 1.74\ e+04\ kg$ service shear load.
- $M_u = 3.81\ e+06\ kg \cdot cm$ ultimate moment.
- $M_s = 2.26\ e+06\ kg \cdot cm$ service moment.

Rigid Upper Plate

From beam:

Flexural strength.		
$F(kg \cdot cm)$	LRFD (ϕM_n)	ASD $(\frac{M_n}{\Omega})$
Nominal moment	4.26e+06	2.84e+06

$F_{ut} = F_{uc} = 1.16\ e+05\ kg$ ultimate tension/compression force to design bending plates (according to M_u).
$F_{st} = F_{sc} = 0.55\ e+05\ kg$ service tension/compression force to design bending plates (according to M_s).

Resultant dimensions:

- $b_p = 15.56cm$ $(6.125in)$ width of plate.
- $t_p = 3.81cm$ $(1.50in)$ thickness of plate.
- $L_p = 40.00cm$ $(16.00in)$ length of plate.
- $\psi = 1.13$ shape factor.

Upper plate nominal force.		
$F(kg)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal yield force	1.35e+05	0.90e+05
Nominal rupture force	1.81e+05	1.21e+05

Fillet Welds for Rigid Plates

Results:

- $EXX/FX = 4900kg/cm^2$ $(70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.80cm$ minimum leg size of weld.
- $tw_{max} = 3.61cm$ maximum leg size of weld.
- $w_p = 1.00cm$ leg size of weld for design of project.
- $Lw_{req} = 74.31cm$ length of weld required.
- $Lw_p = 78.00cm$ length of weld to be adjusted to the project.
- $Lw_{eff} = 78.00cm$ effective length of proportioned weld.
- $R_{nw} = 2078.58kg/cm$ theoretical strength of weld.

Nominal strength of fillet weld.		
$F(kg/cm)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal strength	1558.94	1039.29

Nominal force due to fillet welds.		
$F(kg)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal force	1.22e+05	0.81e+05

Rigid Lower Plate

Results:

- $b_p = 21.59cm$ $(8.50in)$ width of plate.
- $t_p = 3.18cm$ $(1.25in)$ thickness of plate.
- $L_p = 40.00cm$ $(16.00in)$ length of plate.
- $r = 0.917cm$ gyration radius of plate.
- $k = 0.65$ effective length factor.
- $kL_p = 26.00cm$ effective length.
- $\frac{kL_p}{r} = 28.37$ slenderness ratio (adimensional).
- $F_e = 25.02\ e+03\ kg/cm^2$ Euler stress.
- $F_{cr} = 2.43\ e+03\ kg/cm^2$ critical stress.
- $P_n = 1.66\ e+05\ kg$ theoretical compression force resisted.

Compression force resisted by lower plate.		
$F(kg)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal force	1.49e+05	0.99e+05

Shear Design

Beam checking:

- $\frac{h}{t_w}$ = 46.67 slenderness ratio of web.
- k_v = 5.34 (a=0, no transverse stiffeners) adimensional value.
- C_{v1} = 1.00 adimensional value.
- $A_w = 35.84cm^2$ area of the web.

Shear strength of beam section.		
$F(kg)$	LRFD (ϕV_n)	ASD ($\frac{V_n}{\Omega}$)
Nominal shear resistance	6.80e+04	4.53e+04

Shear plates:

- $b_p = 10.16cm$ $(4.00in)$ width of plate.
- $t_p = 0.95cm$ $(0.375in)$ thickness of plate.
- L_p *or* $h = 22.86cm$ $(9.00in)$ length of plate.

Strength of shear plates (the pair).		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal yield strength	5.95e+04	3.96e+04
Nominal rupture strength	8.00e+04	5.33e+04

Results for fillet welds of shear plates:

- $EXX/FX = 4900kg/cm^2$ $(70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.50cm$ minimum leg size of weld.
- $tw_{max} = 0.75cm$ maximum leg size of weld.
- $w_p = 0.50cm$ leg size of weld for design of project.
- $Lw_p = 45.72cm$ length of weld proportioned.
- $Lw_{eff} = 45.72cm$ effective length of proportioned weld.

Nominal strength of fillet welds for shear plates.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal strength	3.56e+04	2.38e+04

Additional quantities.		
Weight	*Length of fillet weld (total)*	*Length of groove weld (total)*
43.64 Kg	45.72 cm of 5mm, and 156.00 cm of 10 mm	37.15 cm

Flanges and Webs with concentrated forces (Chapter J.10)

Flange local bending.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	4.07e+04	2.71e+04
Column	12.37e+04	8.23e+04

From column:

- $t_w = 1.49cm$ thickness of web.
- $k = 3.53cm$ eccentricity of W shape.
- $l_b = 4.50cm$ bearing length (min(t_p,t_{fb})+2w); where:
  - w is the leg size of weld project (w_p).
  - t_p is the thicker thickness of the bending plates.
  - t_{fb} is the thickness of the flange beam.

Web local yielding.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	11.05e+04	7.37e+04

From column:

- $t_f = 2.50cm$ thickness of flange.
- $h = 43.18cm$ depth of W shape.
- $l_b = 4.50cm$ bearing length.
- $Q_f = 1.00$ adimensional factor.

Web local crippling.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	16.64e+04	11.09e+04

Transverse stiffeners (Chapter G2, Section 3 from ANSI/AISC-360-16):

Feature	**Beam**	**Column**
h	40.89cm	43.18cm
t_w	0.88cm	1.49cm
$\frac{h}{t_w}$	46.67	29.06
$2.46\sqrt{\frac{E}{F_{yw}}}$	59.26	59.26
Requirement?	NO	NO

- $F_{yw} = 3515kg/cm^2$ yield stress.
- Transverse stiffeners **not necessary if**:

$$\frac{h}{t_w} < 2.46\sqrt{\frac{E}{F_{yw}}}$$

Stiffening/Continuity Plates

- $C_u = T_u$: 0.00
- A_{req}: $2.35cm^2$
- $\frac{b_{fb}}{3}$: $5.96cm$
- $0.56\sqrt{\frac{E}{F_y}}$: 15.90

For upper plate ($t_{up} = 3.81cm$ $(1.50in)$):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	40.23	34.37	10.56	9.16e+04	6.09e+04
Mid plate	20.11	34.37	5.28	4.58e+04	3.05e+04
Optimal	0.00	34.37	0.00	0.00	0.00

For lower plate ($t_{lp} = 3.18cm$ $(1.25in)$):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	33.53	34.37	10.56	7.63e+04	5.08e+04
Mid plate	16.76	34.37	5.28	3.81e+04	2.54e+04
Optimal	0.00	34.37	0.00	0.00	0.00

Note: strengths (kg) are expressed for just one plate.
For welds (two alternatives, it considers upper and lower edges):

- t_{w1} (weld leg size for alternative 1) = $0.30cm$ $(\frac{1}{8})''$
- t_{w2} (weld leg size for alternative 2) = $0.50cm$

Plate	$LRFD(\phi R_{n1})$	$ASD(\frac{R_{n1}}{\Omega})$	$LRFD(\phi R_{n2})$	$ASD(\frac{R_{n2}}{\Omega})$
FP/MP/OP	3.21e+04	2.14e+04	5.36e+04	3.57e+04

Observations

- Plates have a work performance up to 69% for yield (M_u), and about of 70-86% for ultimate design forces.
- Strength is delimited by resistance of welds.
- There is not need of stiffener elements. The area required for continuity plates is almost negligible.

Finite Element Analysis results: Model 1, Displacements.

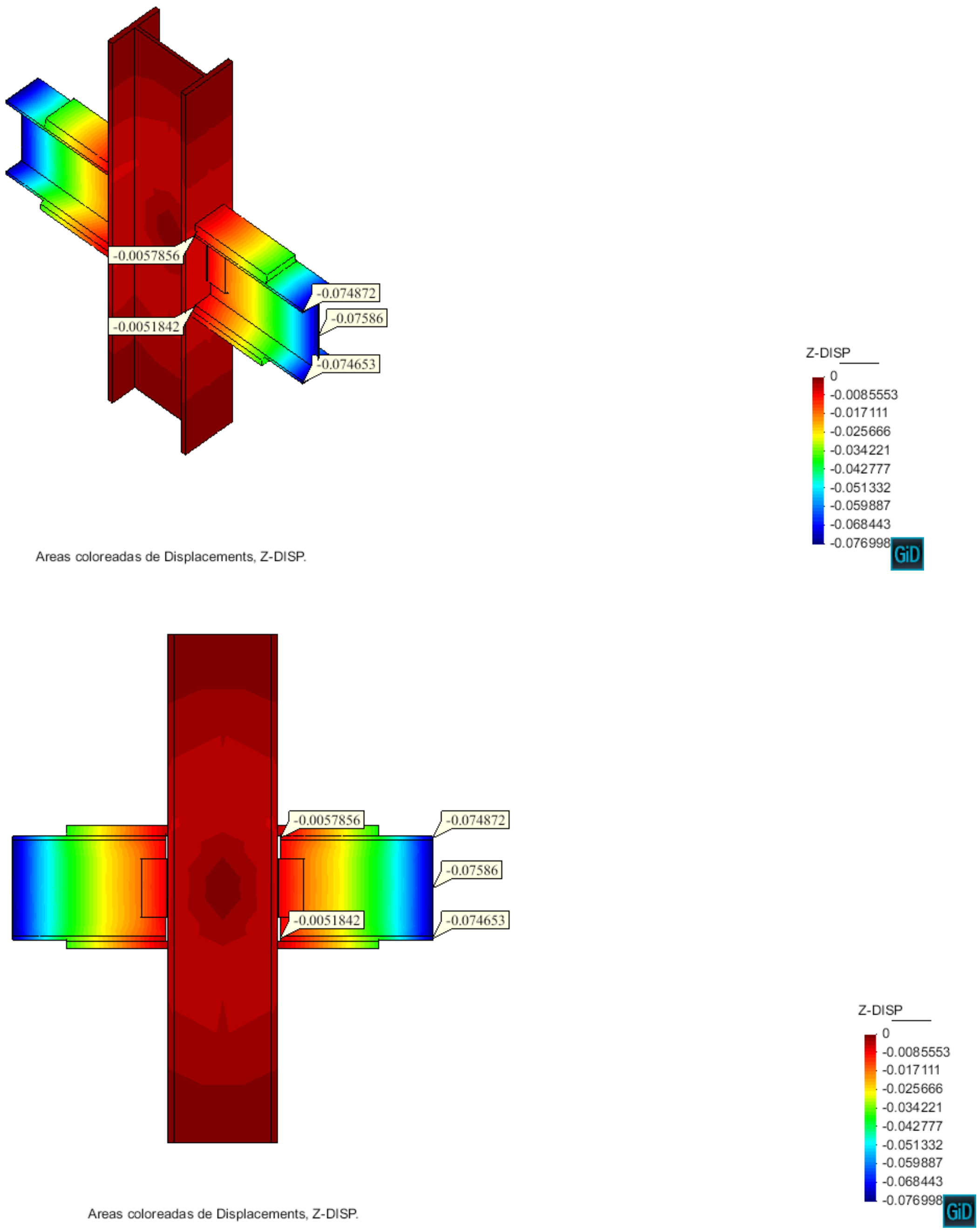


Figure 65: Vertical displacements (Z-axis) in M1-A36 conventional model.

Finite Element Analysis results: Model 1, Tension stresses S_{yy}.

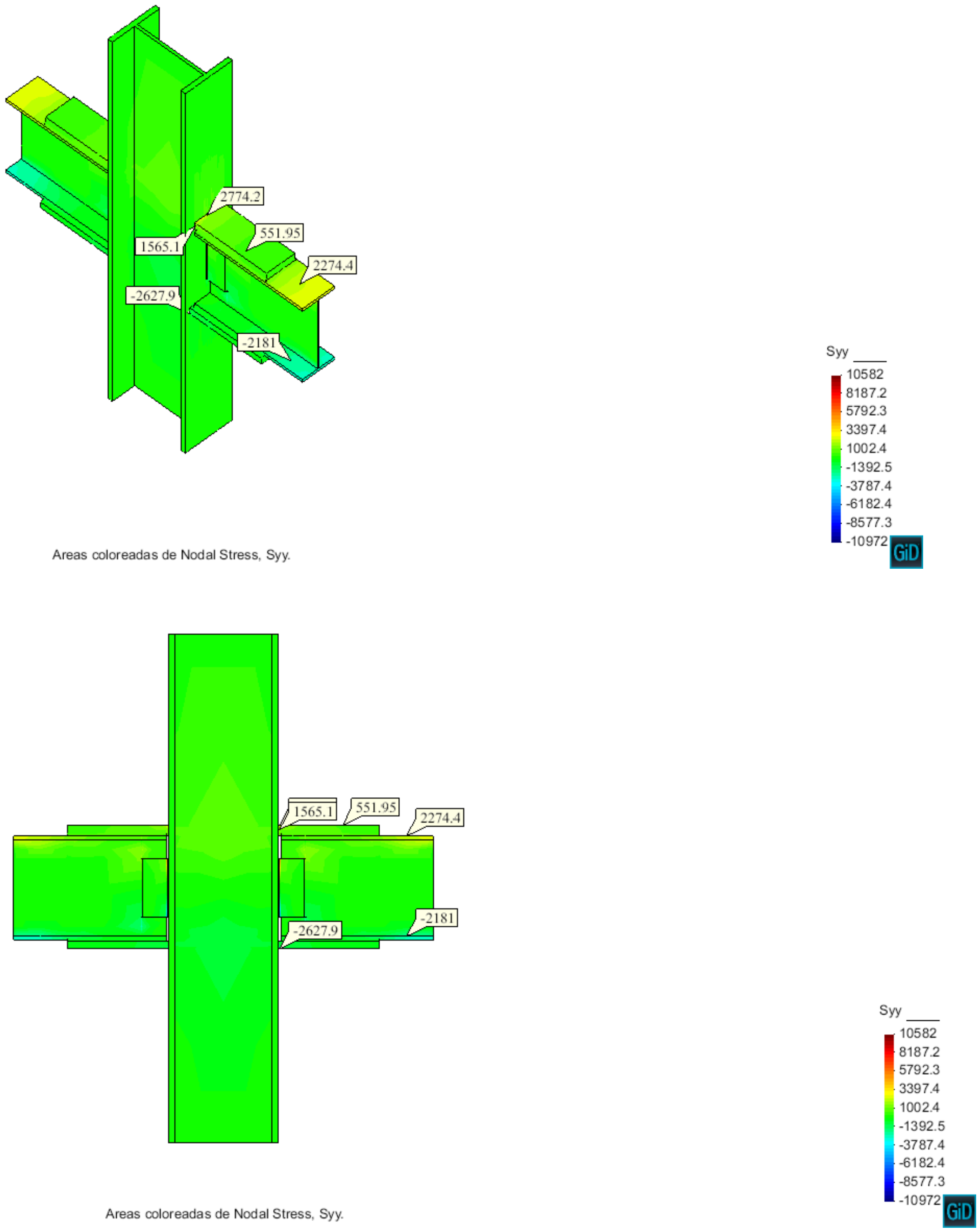
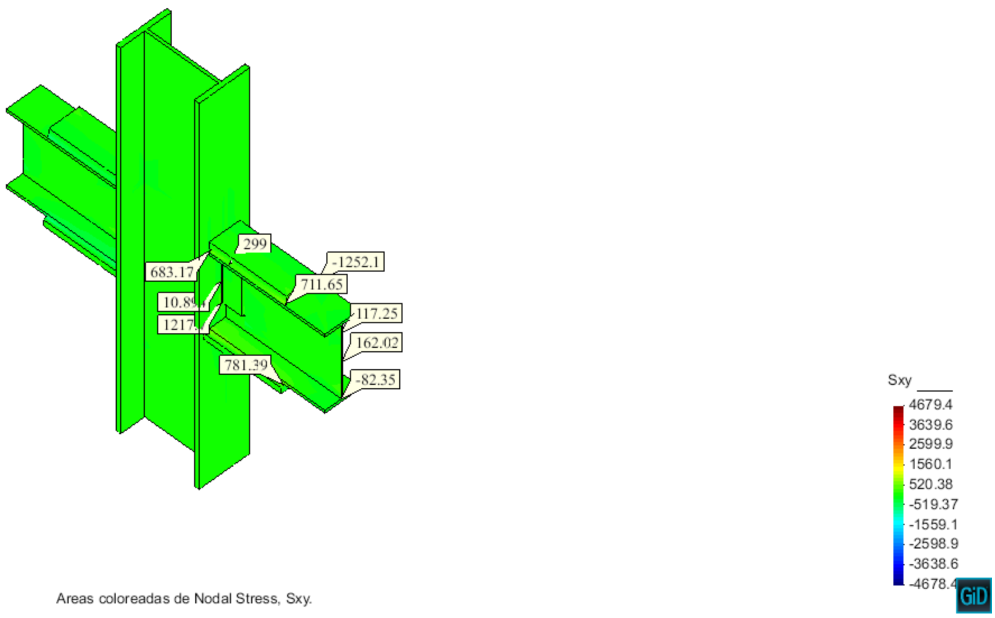


Figure 66: Axial stresses (Y-axis, SYY) in M1-A36 conventional model.

Observations:

- Stresses overcome the yield limit for rigid plates.

Finite Element Analysis results: Model 1, Shear stresses S_{xy}.



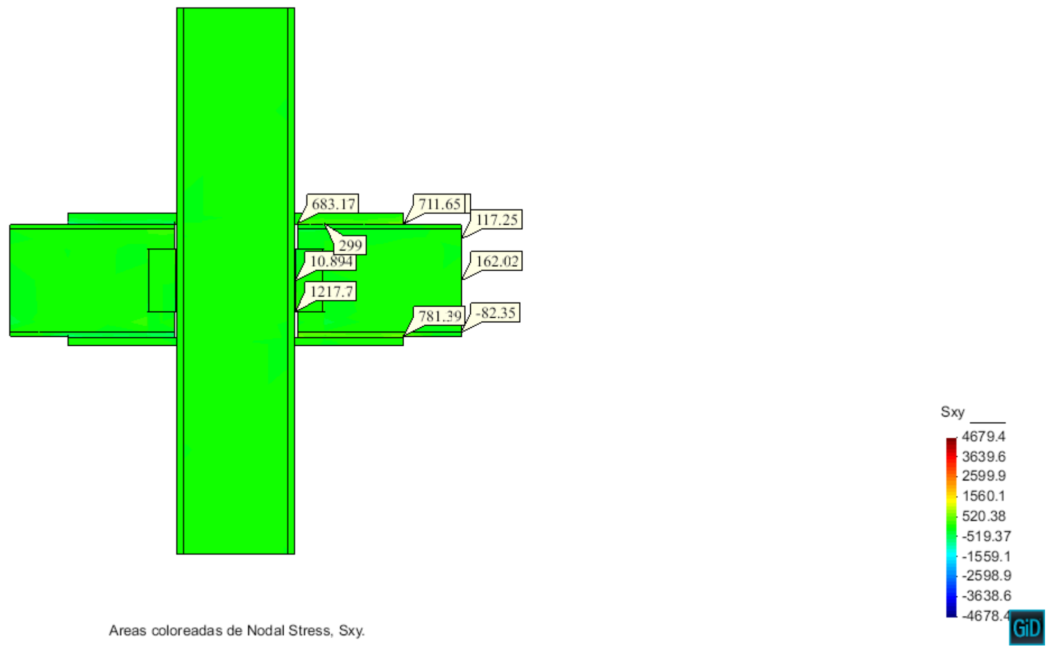


Figure 67: Shear stresses (XY plane, SXY) in M0-A36 conventional model.

Observations:

- All stresses have an acceptable efficiency, even those areas located over the gaps.

Finite Element Analysis results: Model 1, Shear stresses S_{yz}.

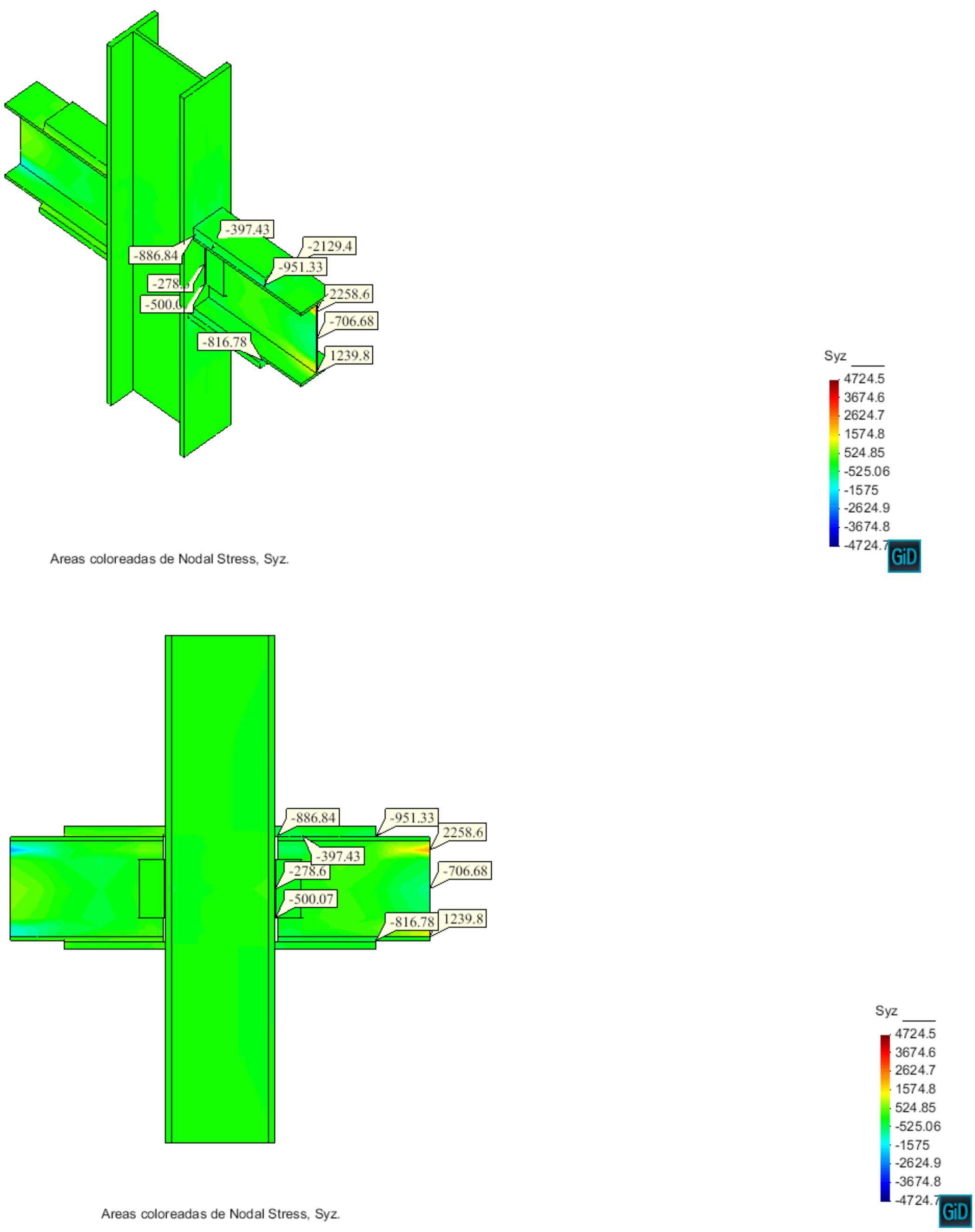


Figure 68: Shear stresses (YZ plane, SYZ) in M1-A36 conventional model.

Observations:

- Just the borders located in gaps concentrate a bit more of tangential stresses.
- Shear stresses concentrations make also a presence in the web and flange transition for the free end of the beam.

Finite Element Analysis results: Model 1, Von Mises stresses VM_s.

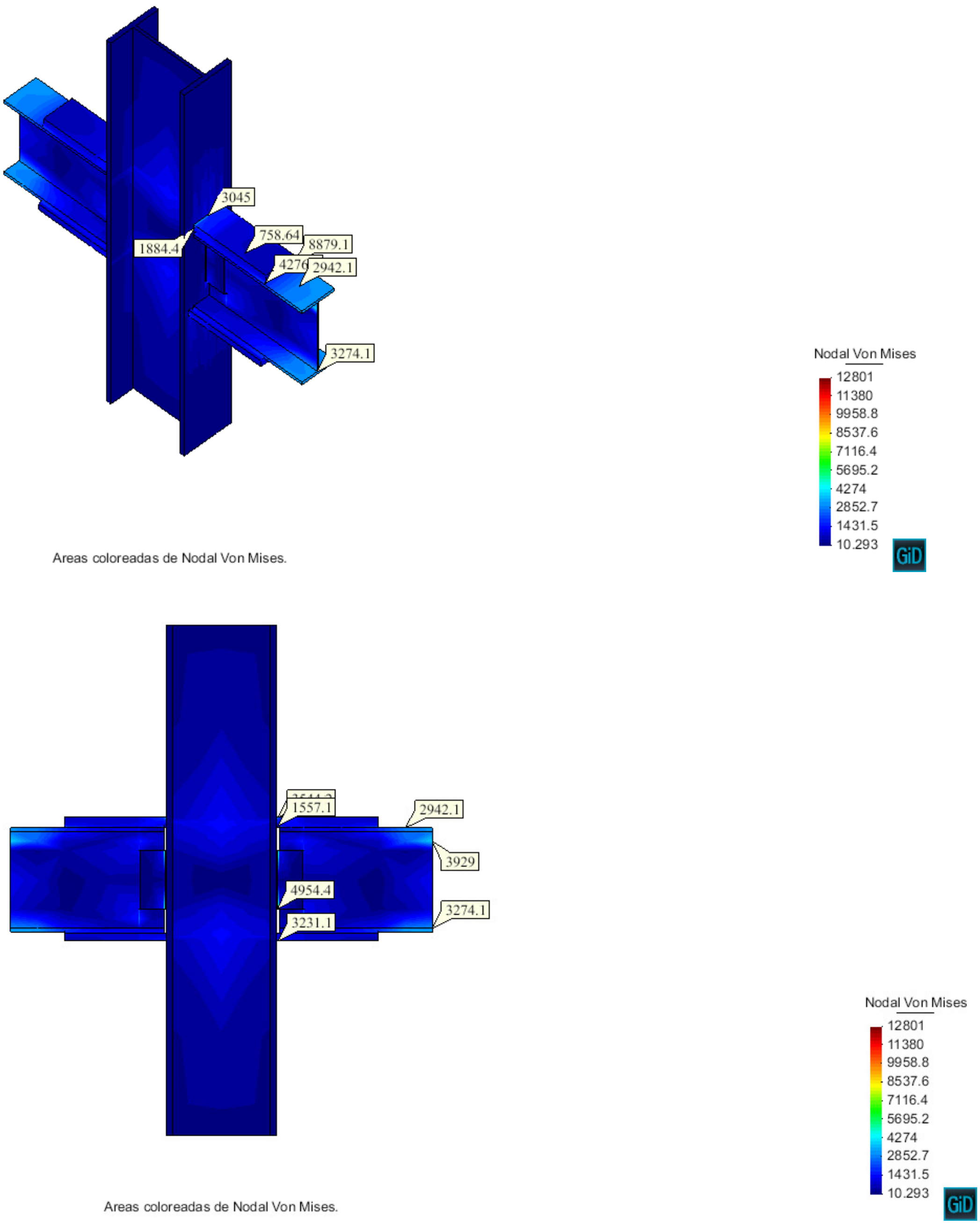


Figure 69: Von Mises stresses in M1-A36 conventional model.

Observations:

- The yield surfaces are born from the bending plates connected to the column flange, the small gaps between welded elements and transitions among web and flanges. Even so, the stresses are under ultimate tensile stress (just plates, not extremities of fillet welds).
- Weld stresses with high values reach failure at the top and bottom edges (located in gaps of fillet welds). According to this, there could be a partial area reduction.

6.1.3 Model 2.

Input Data

Name of model: M2-BM2-CL2-A36, (model for library building)
Beam: (BM_2) W18x46 — Column: (CL_2) W16x100 — Steel Designation: ASTM-A572 Gr. 50
F_y= $3515 kg/cm^2$ $(50 ksi)$ — F_u= $4570 kg/cm^2$ $(65 ksi)$ — Thickness governant: Column
The flange of beam fits in the flange of column $(0.85 b_{fc} > b_{fb})$.
From beam:

- M_{pf}: $3.76\ e+06\ kg \cdot cm$ flanges plastic moment contribution.
- M_{pw}: $1.48\ e+06\ kg \cdot cm$ web plastic moment contribution.
- M_p: $5.24\ e+06\ kg \cdot cm$ total plastic moment.
- M_y: $4.54\ e+06\ kg \cdot cm$ yield moment.
- $\frac{M_{pf}}{M_p} = 71.76\%$ percentage contribution of flanges respect to total plastic moment.
- $\frac{M_{pw}}{M_p} = 28.24\%$ percentage contribution of web respect to total plastic moment.

Design forces:

- $V_u = 2.99\ e+04\ kg$ ultimate shear load.
- $V_s = 2.30\ e+04\ kg$ service shear load.
- $M_u = 3.34\ e+06\ kg \cdot cm$ ultimate moment.
- $M_s = 2.57\ e+06\ kg \cdot cm$ service moment.

Rigid Upper Plate

From beam:

Flexural strength.		
$F(kg \cdot cm)$	LRFD (ϕM_n)	ASD $(\frac{M_n}{\Omega})$
Nominal moment	4.71e+06	3.13e+06

$F_{ut} = F_{uc} = 1.14\ e+05\ kg$ ultimate tension/compression force to design bending plates (according to M_u).
$F_{st} = F_{sc} = 0.56\ e+05\ kg$ service tension/compression force to design bending plates (according to M_s).

Resultant dimensions:

- $b_p = 13.02 cm$ $(5.125 in)$ width of plate.
- $t_p = 4.13 cm$ $(1.625 in)$ thickness of plate.
- $L_p = 40.00 cm$ $(16.00 in)$ length of plate.
- $\psi = 1.15$ shape factor.

Upper plate nominal force.		
$F(kg)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal yield force	1.22e+05	0.81e+05
Nominal rupture force	1.64e+05	1.10e+05

Fillet Welds for Rigid Plates

Results:

- $EXX/FX = 4900kg/cm^2$ $(70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.80cm$ minimum leg size of weld.
- $tw_{max} = 3.93cm$ maximum leg size of weld.
- $w_p = 1.00cm$ leg size of weld for design of project.
- $Lw_{req} = 73.08cm$ length of weld required.
- $Lw_p = 78.00cm$ length of weld to be adjusted to the project.
- $Lw_{eff} = 78.00cm$ effective length of proportioned weld.
- $R_{nw} = 2078.58kg/cm$ theoretical strength of weld.

Nominal strength of fillet weld.		
$F(kg/cm)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal strength	1558.94	1039.29

Nominal force due to fillet welds.		
$F(kg)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal force	1.22e+05	0.81e+05

Rigid Lower Plate

Results:

- $b_p = 17.78cm$ $(7.00in)$ width of plate.
- $t_p = 3.18cm$ $(1.25in)$ thickness of plate.
- $L_p = 40.00cm$ $(16.00in)$ length of plate.
- $r = 0.917cm$ gyration radius of plate.
- $k = 0.65$ effective length factor.
- $kL_p = 26.00cm$ effective length.
- $\frac{kL_p}{r} = 28.37$ slenderness ratio (adimensional).
- $F_e = 25.02\ e+03\ kg/cm^2$ Euler stress.
- $F_{cr} = 2.43\ e+03\ kg/cm^2$ critical stress.
- $P_n = 1.37\ e+05\ kg$ theoretical compression force resisted.

Compression force resisted by lower plate.		
$F(kg)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal force	1.23e+05	0.82e+05

Shear Design

Beam checking:

- $\dfrac{h}{t_w}$ = 50.28 slenderness ratio of web.
- k_v = 5.34 (a=0, no transverse stiffeners) adimensional value.
- C_{v1} = 1.00 adimensional value.
- $A_w = 42.04cm^2$ area of the web.

Shear strength of beam section.		
$F(kg)$	LRFD (ϕV_n)	ASD ($\frac{V_n}{\Omega}$)
Nominal shear resistance	7.98e+04	5.31e+04

Shear plates:

- $b_p = 10.16cm$ $(4.00in)$ width of plate.
- $t_p = 0.95cm$ $(0.375in)$ thickness of plate.
- L_p *or* $h = 22.86cm$ $(9.00in)$ length of plate.

Strength of shear plates (the pair).		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal yield strength	5.95e+04	3.96e+04
Nominal rupture strength	7.99e+04	5.33e+04

Results for fillet welds of shear plates:

- $EXX/FX = 4900kg/cm^2$ $(70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.50cm$ minimum leg size of weld.
- $tw_{max} = 0.75cm$ maximum leg size of weld.
- $w_p = 0.50cm$ leg size of weld for design of project.
- $Lw_p = 45.72cm$ length of weld proportioned.
- $Lw_{eff} = 45.72cm$ effective length of proportioned weld.

Nominal strength of fillet welds for shear plates.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal strength	3.56e+04	2.38e+04

Additional quantities.		
Weight	*Length of fillet weld (total)*	*Length of groove weld (total)*
38.10 Kg	45.72 cm of 5 mm, and 156.00 cm of 10 mm	30.80 cm

Flanges and Webs with concentrated forces (Chapter J.10)

Flange local bending.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	4.67e+04	3.11e+04
Column	12.37e+04	8.23e+04

From column:

- $t_w = 1.49cm$ thickness of web.
- $k = 3.53cm$ eccentricity of W shape.
- $l_b = 4.50cm$ bearing length (min(t_p,t_{fb})+2w); where:
  - w is the leg size of weld project (w_p).
  - t_p is the thicker thickness of the bending plates.
  - t_{fb} is the thickness of the flange beam.

Web local yielding.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	11.05e+04	7.37e+04

From column:

- $t_f = 2.50cm$ thickness of flange.
- $h = 43.18cm$ depth of W shape.
- $l_b = 4.50cm$ bearing length.
- $Q_f = 1.00$ adimensional factor.

Web local crippling.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	16.64e+04	11.09e+04

Transverse stiffeners (Chapter G2, Section 3 from ANSI/AISC-360-16):

Feature	**Beam**	**Column**
h	45.97cm	43.18cm
t_w	0.91cm	1.49cm
$\frac{h}{t_w}$	50.28	29.06
$2.46\sqrt{\frac{E}{F_{yw}}}$	59.26	59.26
Requirement?	NO	NO

- $F_{yw} = 3515kg/cm^2$ yield stress.
- Transverse stiffeners **not necessary if**:

$$\frac{h}{t_w} < 2.46\sqrt{\frac{E}{F_{yw}}}$$

Stiffening/Continuity Plates

- $C_u = T_u$: 0.00
- A_{req}: $1.51cm^2$
- $\frac{b_{fb}}{3}$: $5.13cm$
- $0.56\sqrt{\frac{E}{F_y}}$: 15.90

For upper plate ($t_{up} = 4.13cm$ ($1.625in$)):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	46.21	34.37	10.56	10.52e+04	7.00e+04
Mid plate	23.10	34.37	5.28	5.26e+04	3.50e+04
Optimal	0.00	34.37	0.00	0.00	0.00

For lower plate ($t_{lp} = 3.18cm$ ($1.25in$)):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	35.54	34.37	10.56	8.09e+04	5.38e+04
Mid plate	17.77	34.37	5.28	4.05e+04	2.69e+04
Optimal	0.00	34.37	0.00	0.00	0.00

Note: strengths (kg) are expressed for just one plate.
For welds (two alternatives, it considers upper and lower edges):

- t_{w1} (weld leg size for alternative 1) = $0.30cm$ $(\frac{1}{8})''$
- t_{w2} (weld leg size for alternative 2) = $0.50cm$

Plate	$LRFD(\phi R_{n1})$	$ASD(\frac{R_{n1}}{\Omega})$	$LRFD(\phi R_{n2})$	$ASD(\frac{R_{n2}}{\Omega})$
FP/MP/OP	3.21e+04	2.14e+04	5.36e+04	3.57e+04

Observations

- Plates have a work performance up to 59% for yield (M_u). Both plates and welds reach failure (yield) simultaneously for ultimate design forces.
- There is not need of stiffener elements. The area required for continuity plates is almost negligible.
- The upper plate has an efficiency of 93%, although, a greater thickness might get constructive complications for welding to the column flange. The lower plate works at a 83%, both plates considering plastic moment design.

Finite Element Analysis results: Model 2, Displacements.

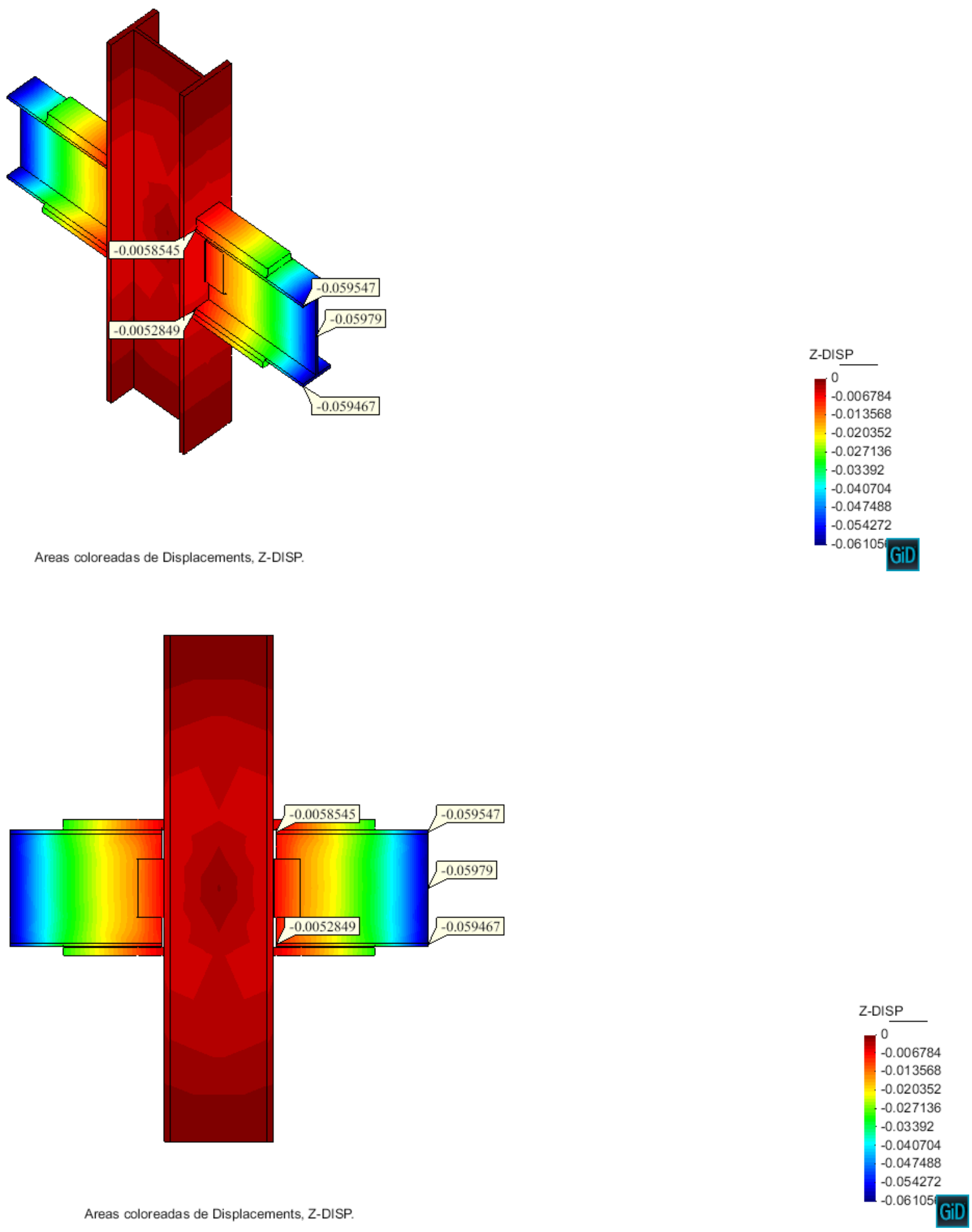
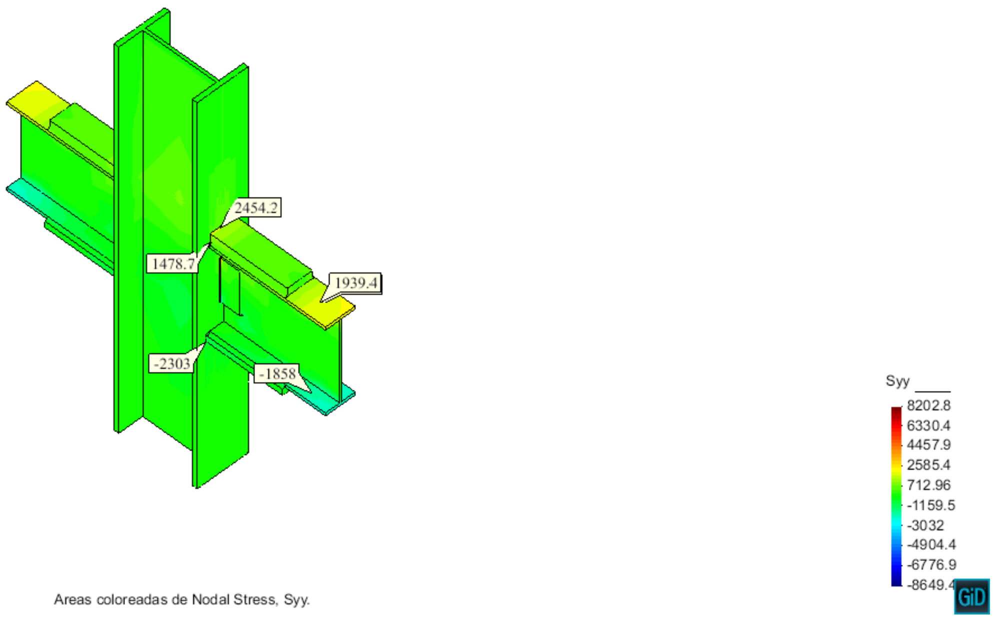


Figure 70: Vertical displacements (Z-axis) in M2-A36 conventional model.

Finite Element Analysis results: Model 2, Tension stresses S_{yy}.



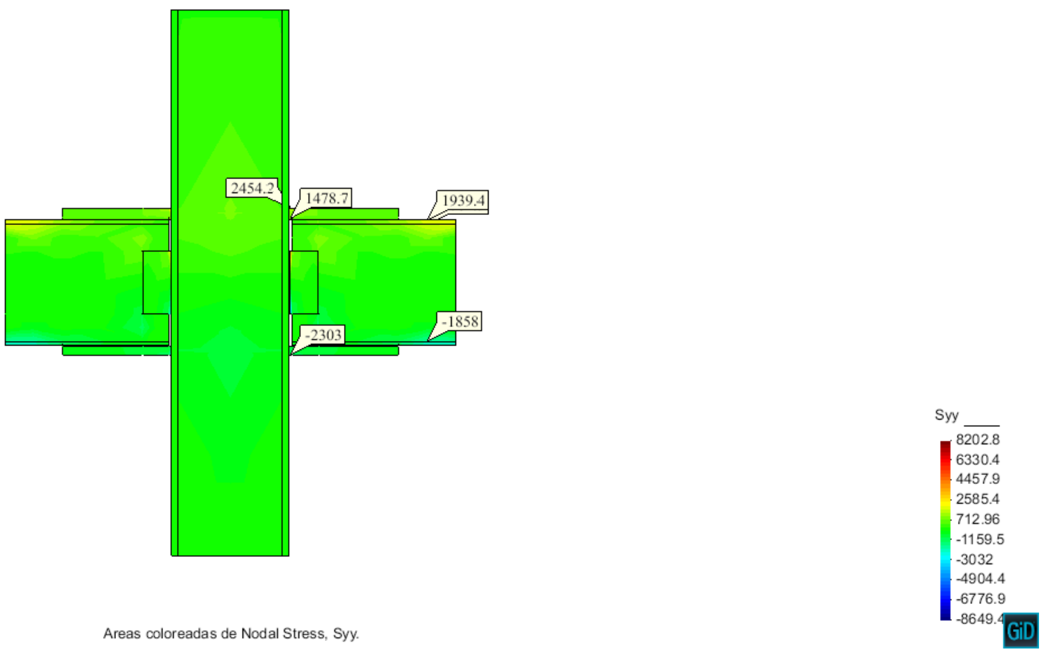


Figure 71: Axial stresses (Y-axis, SYY) in M2-A36 conventional model.

Observations:

- The compression plates works under the limit yield stress, as the same way the tension plate does.

Finite Element Analysis results: Model 2, Shear stresses S_{xy}.

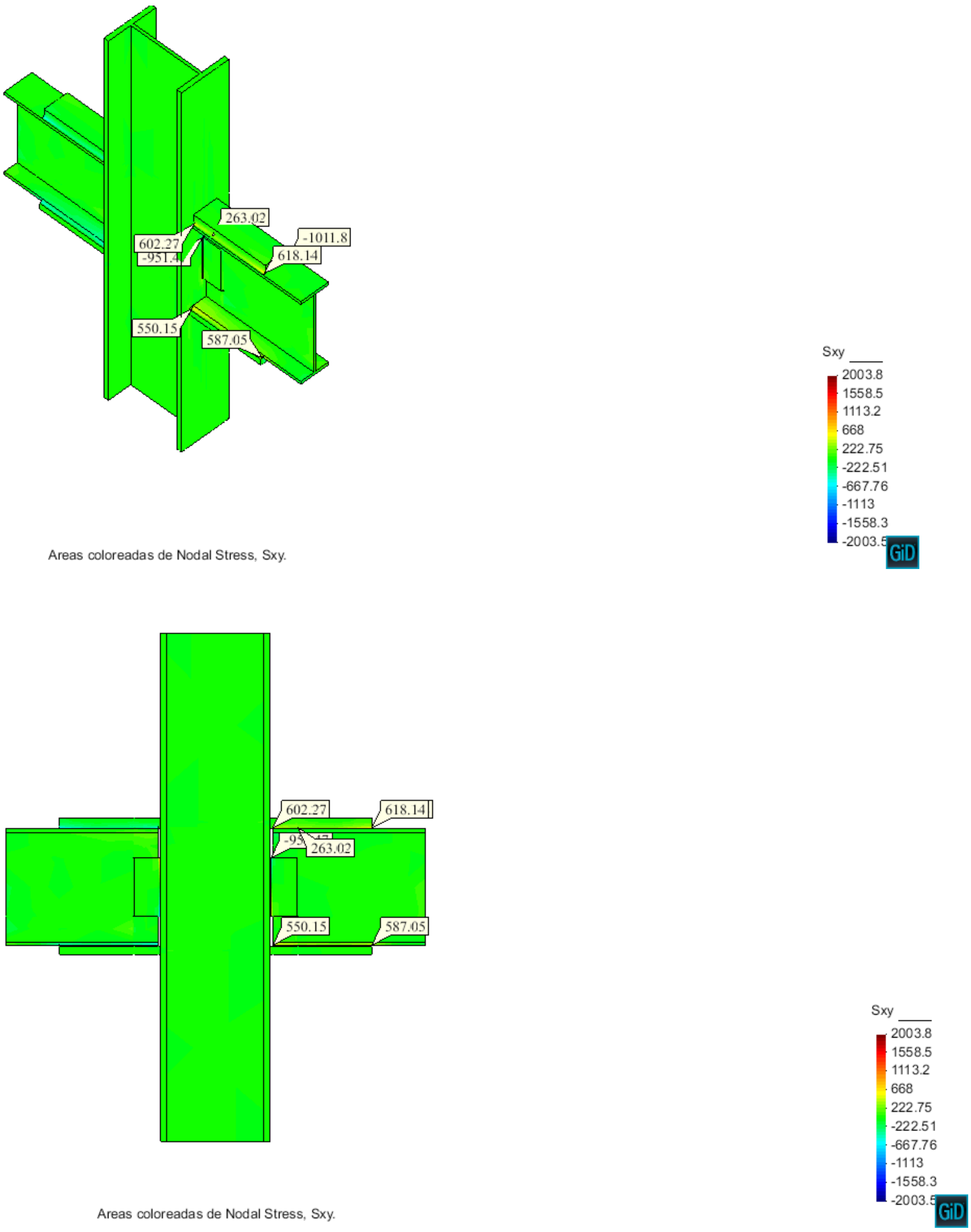


Figure 72: Shear stresses (XY plane, SXY) in M2-A36 conventional model.

Observations:

- The values are below the theoretical and nominal strengths, so, fillet welds have a good performance.

Finite Element Analysis results: Model 2, Shear stresses S_{yz}.

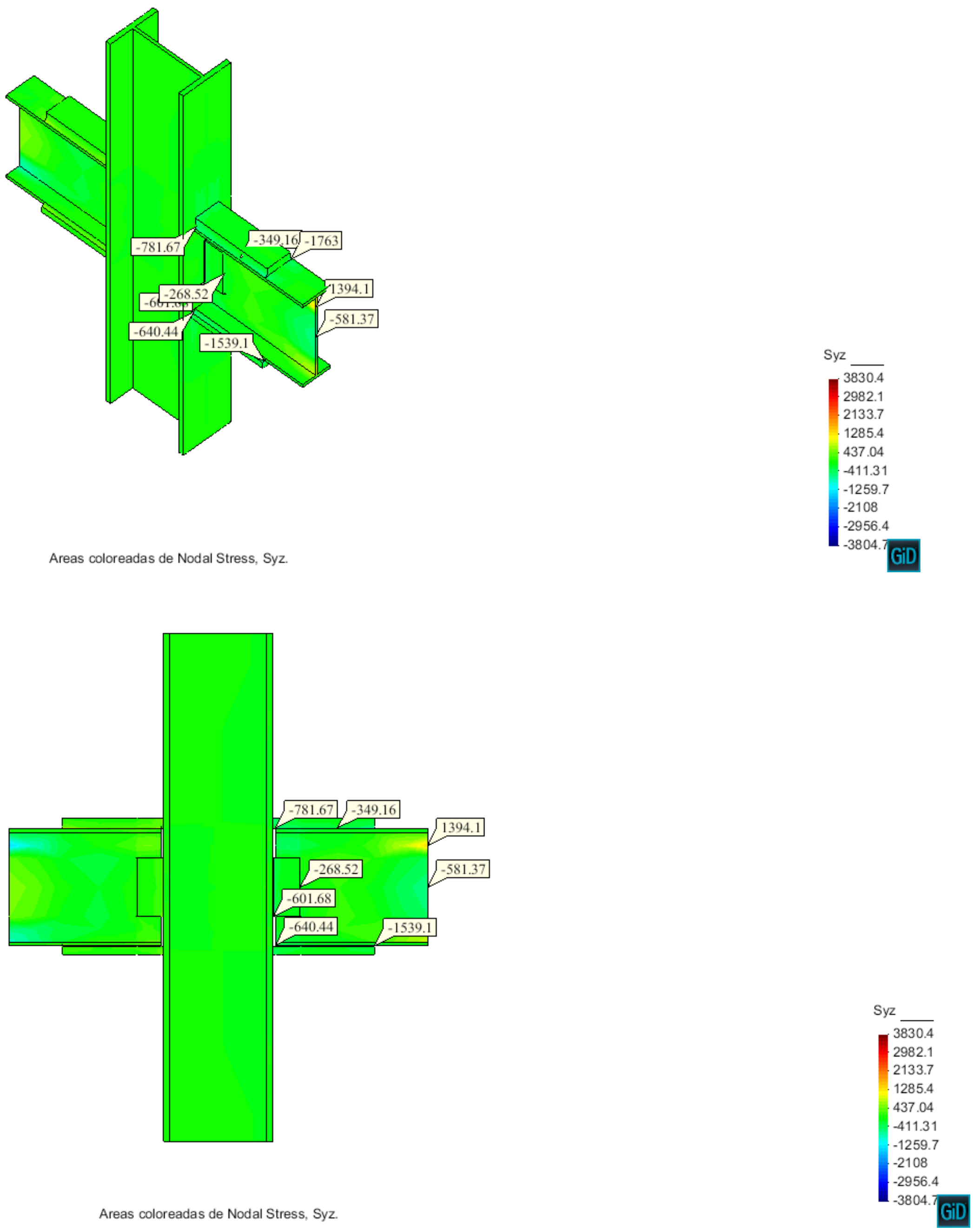
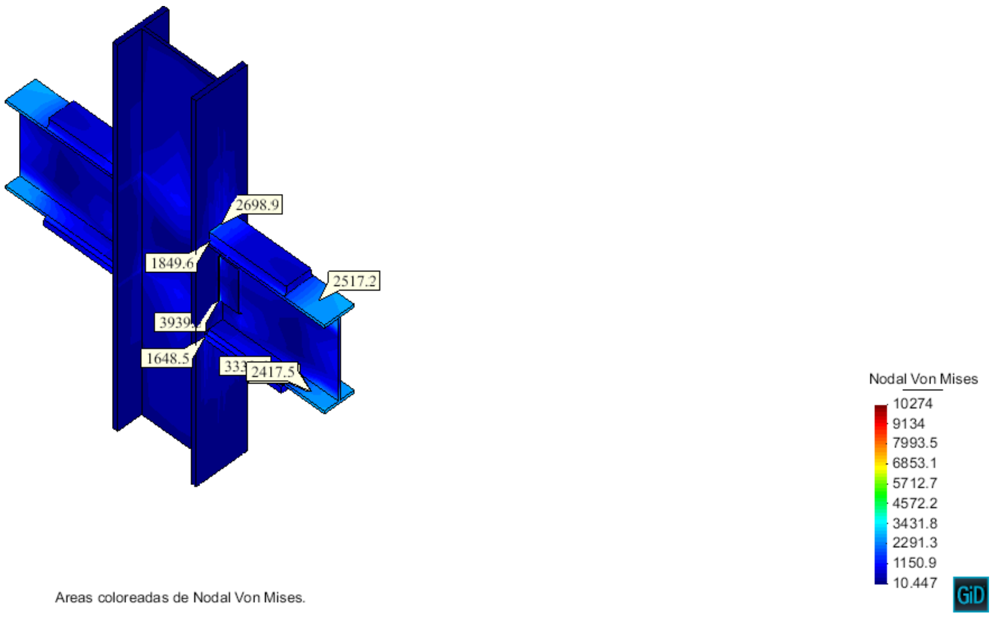


Figure 73: Shear stresses (YZ plane, SYZ) in M2-A36 conventional model.

Observations:

- The values are below the theoretical and nominal strengths, just the roots of the longitudinal upper fillet welds, located at the top of the connection arm concentrate around of 12% more than their nominal strength.

Finite Element Analysis results: Model 2, Von Mises stresses VM_s.



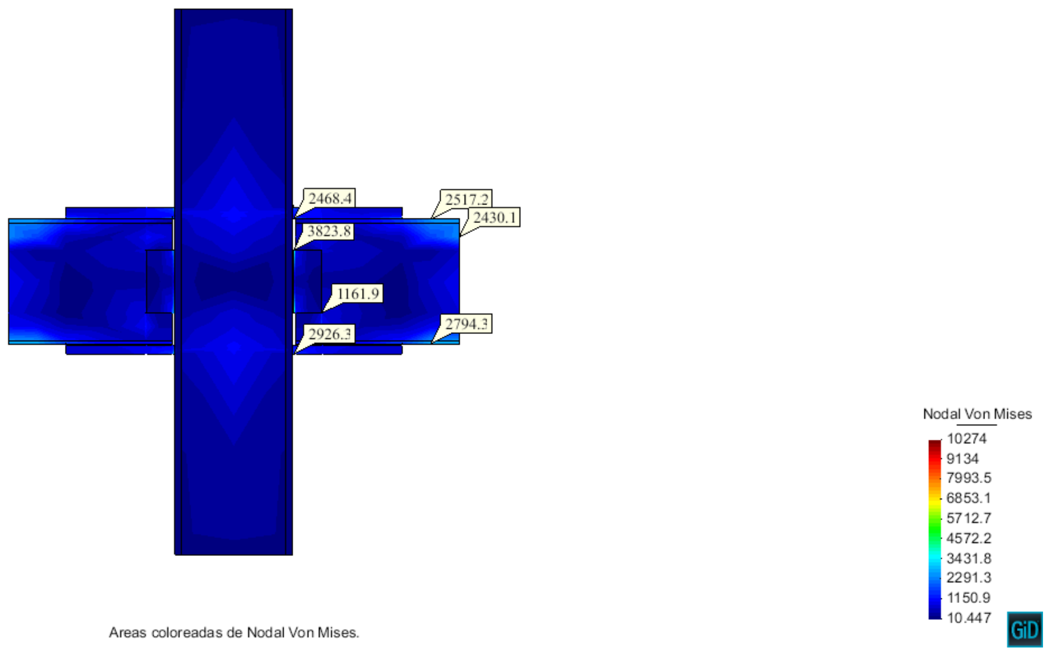


Figure 74: Von Mises stresses in M2-A36 conventional model.

Observations:

- Yield surfaces appear around of the groove weld areas, and the extreme edges of fillet welds.

6.1.4 Model 3.

Input Data

Name of model: M3-BM3-CL3-A36, (arbitrary model)
Beam: (BM_3) W24x55 — Column: (CL_3) W16x100 — Steel Designation: ASTM-A572 Gr. 50
F_y= $3515kg/cm^2$ $(50ksi)$ — F_u= $4570kg/cm^2$ $(65ksi)$ — Thickness governant: Column
The flange of beam fits in the flange of column $(0.85b_{fc} > b_{fb})$.
From beam:

- M_{pf}: $4.76\ e + 06\ kg \cdot cm$ flanges plastic moment contribution.
- M_{pw}: $2.90\ e + 06\ kg \cdot cm$ web plastic moment contribution.
- M_p: $7.66\ e + 06\ kg \cdot cm$ total plastic moment.
- M_y: $6.57\ e + 06\ kg \cdot cm$ yield moment.
- $\frac{M_{pf}}{M_p} = 62.12\%$ percentage contribution of flanges respect to total plastic moment.
- $\frac{M_{pw}}{M_p} = 37.88\%$ percentage contribution of web respect to total plastic moment.

Design forces:

- $V_u = 3.77\ e + 04\ kg$ ultimate shear load.
- $V_s = 2.69\ e + 04\ kg$ service shear load.
- $M_u = 5.25\ e + 06\ kg \cdot cm$ ultimate moment.
- $M_s = 3.75\ e + 06\ kg \cdot cm$ service moment.

Rigid Upper Plate

From beam:

Flexural strength.		
$F(kg \cdot cm)$	LRFD (ϕM_n)	ASD $(\frac{M_n}{\Omega})$
Nominal moment	6.90e+06	4.59e+06

$F_{ut} = F_{uc} = 1.28\ e+05\ kg$ ultimate tension/compression force to design bending plates (according to M_u).
$F_{st} = F_{sc} = 0.63\ e + 05\ kg$ service tension/compression force to design bending plates (according to M_s).

Resultant dimensions:

- $b_p = 15.56cm$ $(6.125in)$ width of plate.
- $t_p = 4.13cm$ $(1.625in)$ thickness of plate.
- $L_p = 45.00cm$ $(18.00in)$ length of plate.
- $\psi = 1.18$ shape factor.

Upper plate nominal force.		
$F(kg)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal yield force	1.46e+05	0.97e+05
Nominal rupture force	1.96e+05	1.31e+05

Fillet Welds for Rigid Plates

Results:

- $EXX/FX = 4900kg/cm^2$ $(70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.80cm$ minimum leg size of weld.
- $tw_{max} = 3.93cm$ maximum leg size of weld.
- $w_p = 1.00cm$ leg size of weld for design of project.
- $Lw_{req} = 82.01cm$ length of weld required.
- $Lw_p = 88.00cm$ length of weld to be adjusted to the project.
- $Lw_{eff} = 88.00cm$ effective length of proportioned weld.
- $R_{nw} = 2078.58kg/cm$ theoretical strength of weld.

Nominal strength of fillet weld.		
$F(kg/cm)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal strength	1558.94	1039.29

Nominal force due to fillet welds.		
$F(kg)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal force	1.37e+05	0.91e+05

Rigid Lower Plate

Results:

- $b_p = 21.59cm$ $(8.50in)$ width of plate.
- $t_p = 3.18cm$ $(1.25in)$ thickness of plate.
- $L_p = 45.00cm$ $(18.00in)$ length of plate.
- $r = 0.917cm$ gyration radius of plate.
- $k = 0.65$ effective length factor.
- $kL_p = 29.25cm$ effective length.
- $\frac{kL_p}{r} = 31.91$ slenderness ratio (adimensional).
- $F_e = 19.77\ e+03\ kg/cm^2$ Euler stress.
- $F_{cr} = 2.40\ e+03\ kg/cm^2$ critical stress.
- $P_n = 1.64\ e+05\ kg$ theoretical compression force resisted.

Compression force resisted by lower plate.		
$F(kg)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal force	1.48e+05	0.98e+05

Shear Design

Beam checking:

- $\frac{h}{t_w}$ = 59.75 slenderness ratio of web.
- k_v = 5.34 (a=0, no transverse stiffeners) adimensional value.
- C_{v1} = 1.00 adimensional value.
- $A_w = 60.14cm^2$ area of the web.

Shear strength of beam section.		
$F(kg)$	LRFD (ϕV_n)	ASD ($\frac{V_n}{\Omega}$)
Nominal shear resistance	11.42e+04	7.59e+04

Shear plates:

- $b_p = 10.16cm$ $(4.00in)$ width of plate.
- $t_p = 0.95cm$ $(0.375in)$ thickness of plate.
- L_p *or* $h = 35.56cm$ $(14.00in)$ length of plate.

Strength of shear plates (the pair).		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal yield strength	9.25e+04	6.16e+04
Nominal rupture strength	12.43e+04	8.29e+04

Results for fillet welds of shear plates:

- $EXX/FX = 4900kg/cm^2$ $(70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.50cm$ minimum leg size of weld.
- $tw_{max} = 0.75cm$ maximum leg size of weld.
- $w_p = 0.625cm$ leg size of weld for design of project.
- $Lw_p = 71.12cm$ length of weld proportioned.
- $Lw_{eff} = 69.16cm$ effective length of proportioned weld.

Nominal strength of fillet welds for shear plates.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal strength	6.38e+04	4.25e+04

Additional quantities.		
Weight	*Length of fillet weld (total)*	*Length of groove weld (total)*
52.34 Kg	71.12 cm of 6.3 mm, and 176.00 cm of 10 mm	37.15 cm

Flanges and Webs with concentrated forces (Chapter J.10)

Flange local bending.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	3.25e+04	2.16e+04
Column	12.37e+04	8.23e+04

From column:

- t_w = $1.49cm$ thickness of web.
- k = $3.53cm$ eccentricity of W shape.
- l_b = $4.50cm$ bearing length (min(t_p,t_{fb})+2w); where:
  - w is the leg size of weld project (w_p).
  - t_p is the thicker thickness of the bending plates.
  - t_{fb} is the thickness of the flange beam.

Web local yielding.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	11.05e+04	7.37e+04

From column:

- t_f = $2.50cm$ thickness of flange.
- h = $43.18cm$ depth of W shape.
- l_b = $4.50cm$ bearing length.
- Q_f = 1.00 adimensional factor.

Web local crippling.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	16.64e+04	11.09e+04

Transverse stiffeners (Chapter G2, Section 3 from ANSI/AISC-360-16):

Feature	**Beam**	**Column**
h	59.94cm	43.18cm
t_w	0.91cm	1.49cm
$\frac{h}{t_w}$	59.75	29.06
$2.46\sqrt{\frac{E}{F_{yw}}}$	59.26	59.26
Requirement?	YES	NO

- F_{yw} = $3515kg/cm^2$ yield stress.
- Transverse stiffeners **not necessary if**:

$$\frac{h}{t_w} < 2.46\sqrt{\frac{E}{F_{yw}}}$$

Stiffening/Continuity Plates

- $C_u = T_u$: 0.00
- A_{req}: $7.62cm^2$
- $\frac{b_{fb}}{3}$: $5.94cm$
- $0.56\sqrt{\frac{E}{F_y}}$: 15.90

For upper plate ($t_{up} = 4.13cm$ ($1.625in$)):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	43.59	34.37	10.56	9.92e+04	6.60e+04
Mid plate	21.79	34.37	5.28	4.96e+04	3.30e+04
Optimal	8.26	34.37	2.00	1.89e+04	1.25e+04

For lower plate ($t_{lp} = 2.22cm$ ($0.875in$)):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	33.53	34.37	10.56	7.63e+04	5.08e+04
Mid plate	16.76	34.37	5.28	3.81e+04	2.54e+04
Optimal	9.53	34.37	3.00	2.17e+04	1.44e+04

Note: strengths (kg) are expressed for just one plate.
For welds (two alternatives, it considers upper and lower edges):

- t_{w1} (weld leg size for alternative 1) = $0.30cm$ $(\frac{1}{8})''$
- t_{w2} (weld leg size for alternative 2) = $0.50cm$

Plate	$LRFD(\phi R_{n1})$	$ASD(\frac{R_{n1}}{\Omega})$	$LRFD(\phi R_{n2})$	$ASD(\frac{R_{n2}}{\Omega})$
FP/MP/OP	3.21e+04	2.14e+04	5.36e+04	3.57e+04

Observations

- Plates have a work performance up to 60% for yield (M_u), and about of 77-88% for ultimate design forces.
- Strength is delimited by resistance of welds.
- A transverse stiffener is required (seated on beam). The area required for continuity plates can be adjusted with a minor thickness (it has to accomplish the allowable width), the chosen one was taken according to geometric conditions programmed for the spreadsheet and problem type code.

Finite Element Analysis results: Model 3, Dispacements.

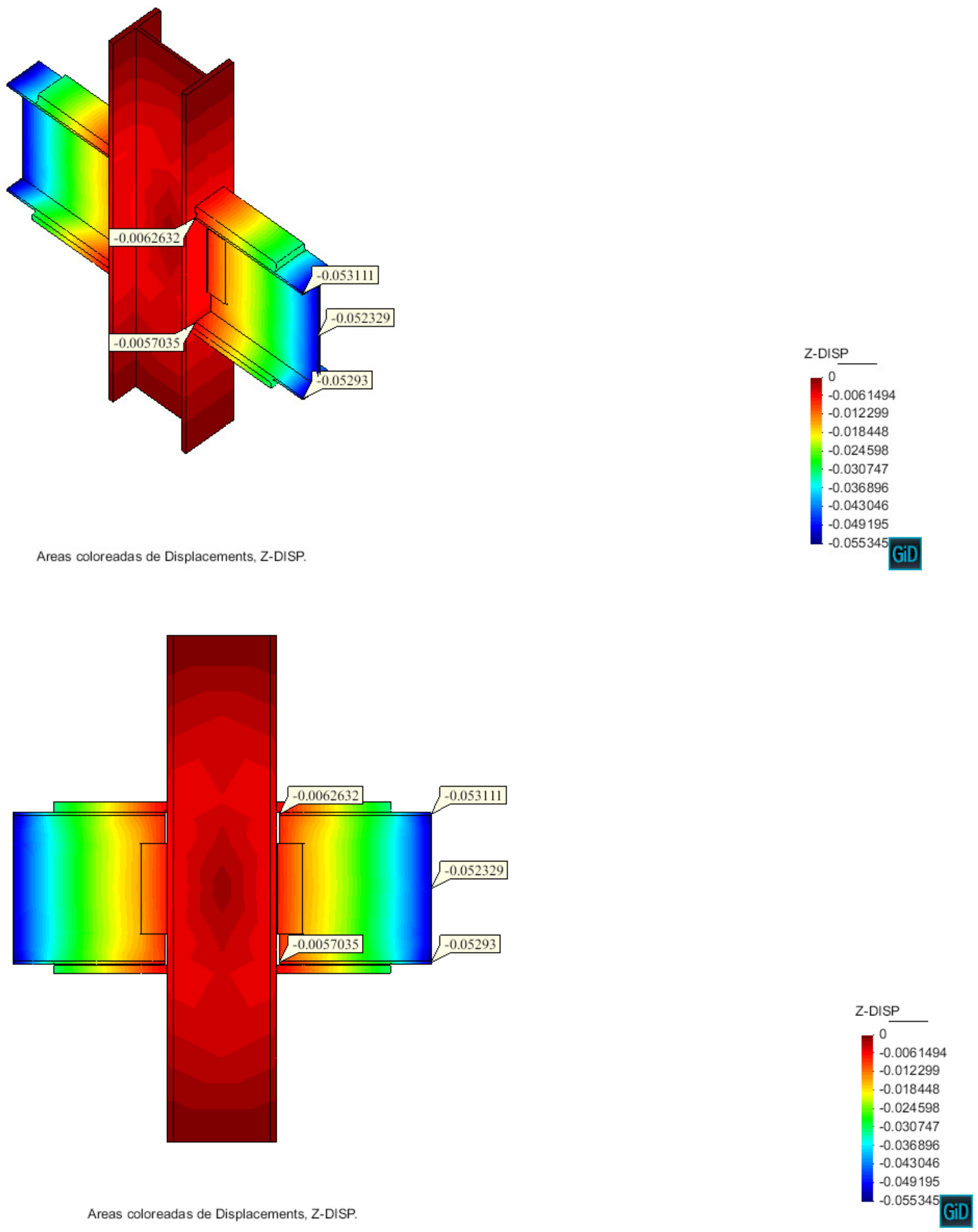


Figure 75: Vertical displacements (Z-axis) in M3-A36 conventional model.

Finite Element Analysis results: Model 3, Tension stresses S_{yy}.

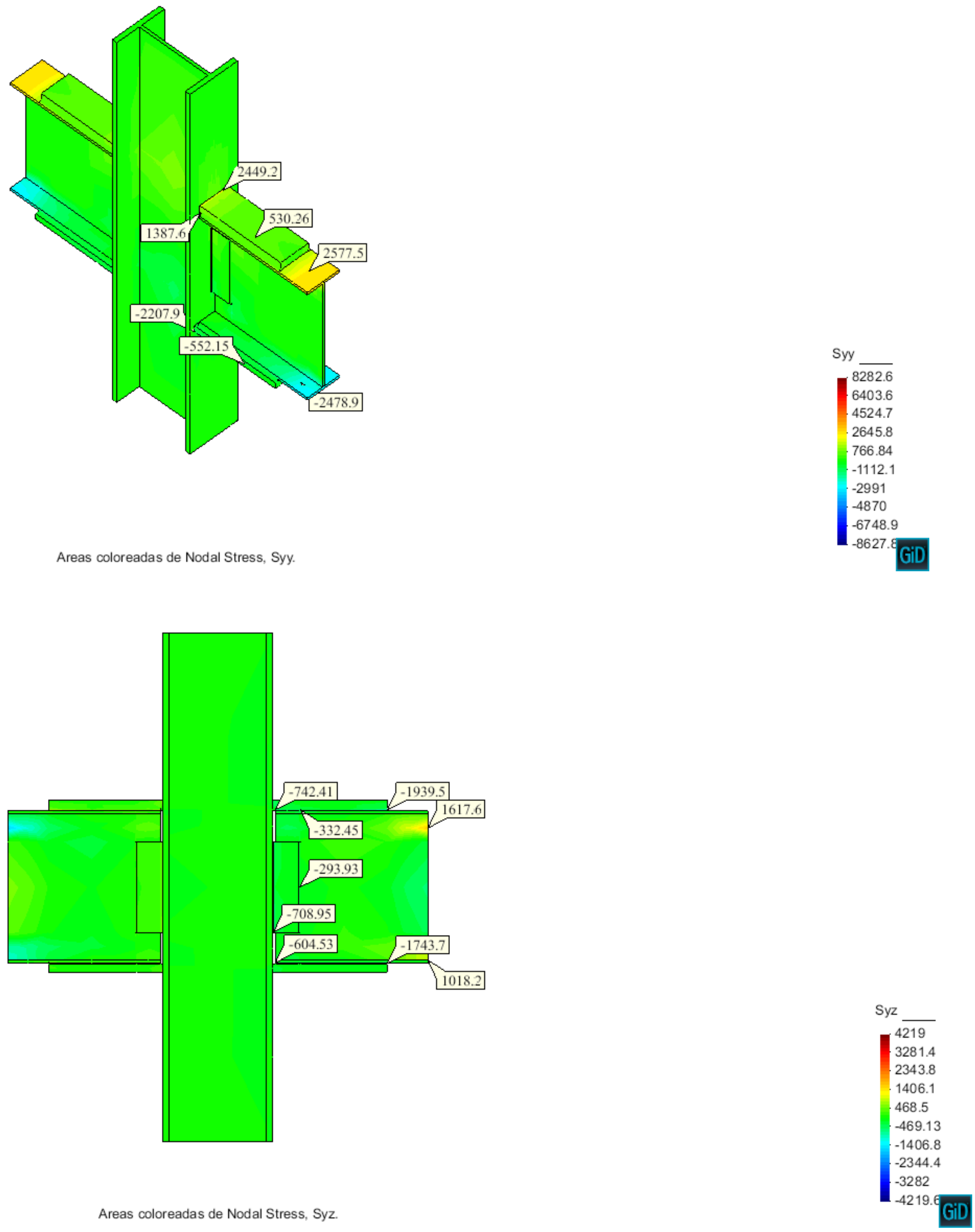
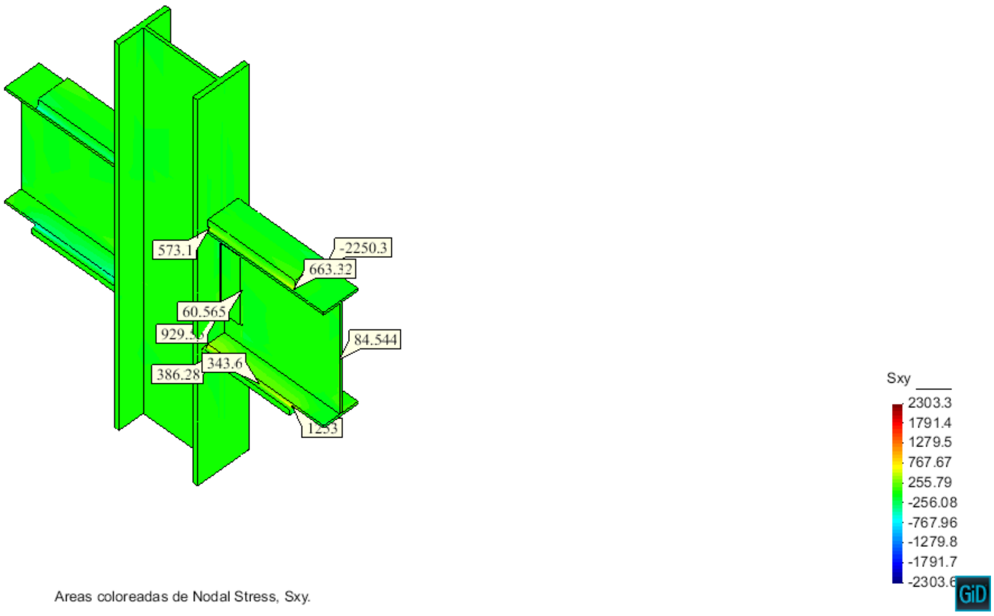


Figure 76: Axial stresses (Y-axis, SYY) in M3-A36 conventional model.

Observations:

- Plates subjected to flexure maintain an axial stress under the limit yield stress.

Finite Element Analysis results: Model 3, Shear stresses S_{xy}.



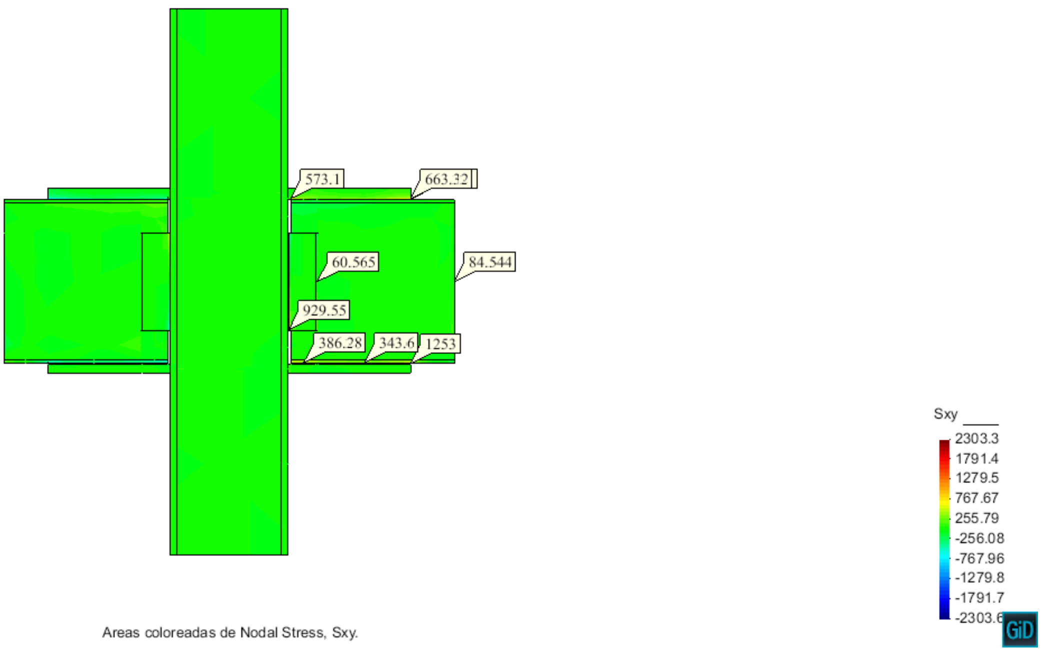


Figure 77: Shear stresses (XY plane, SXY) in M3-A36 conventional model.

Observations:

- Extremities of longitudinal fillet welds reach a big concentration on tangential stress above 10% for their theoretical strength, and 30% for restricted under ANSI-AISC 360-16 specifications.

Finite Element Analysis results: Model 3, Shear stresses S_{yz}.

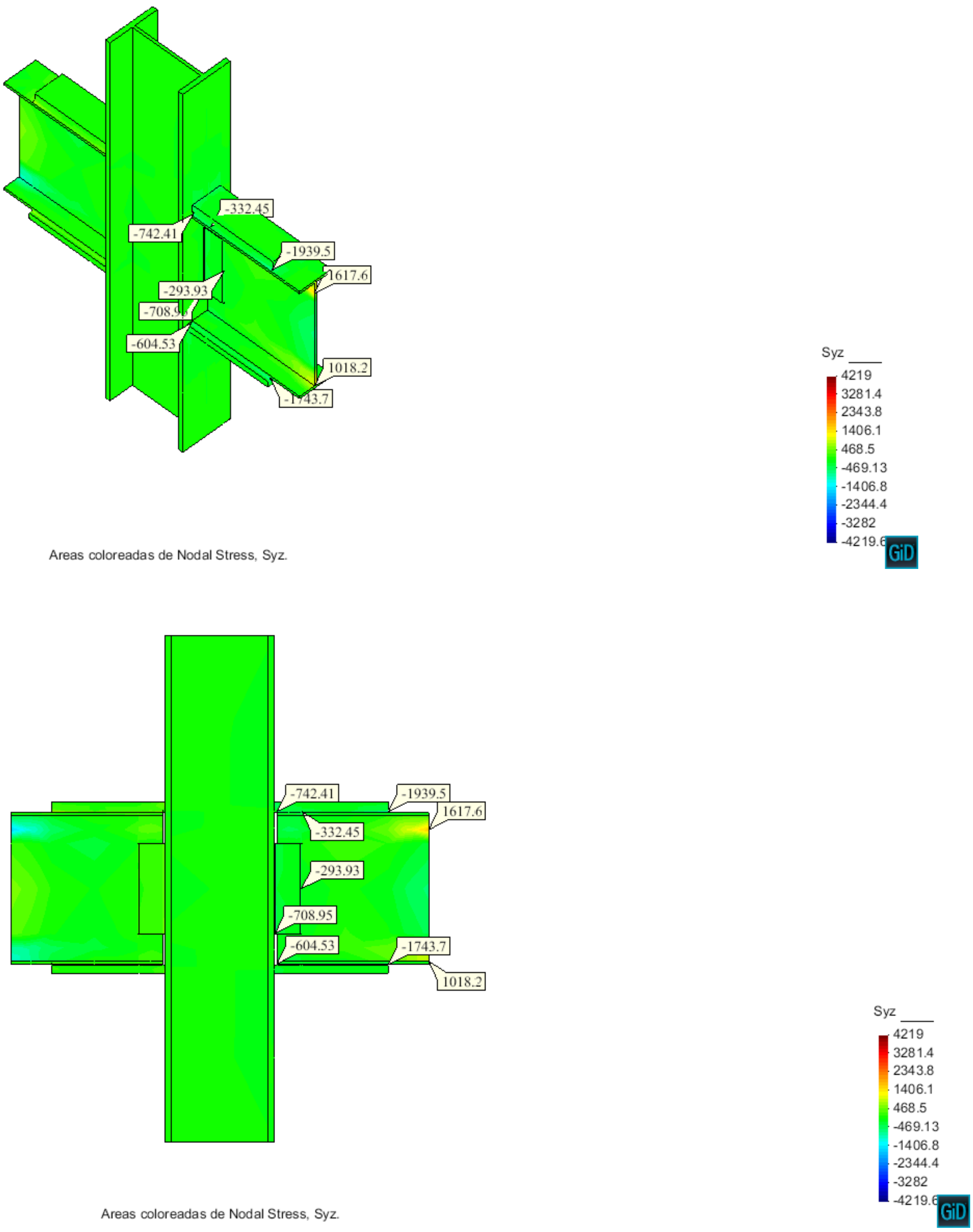
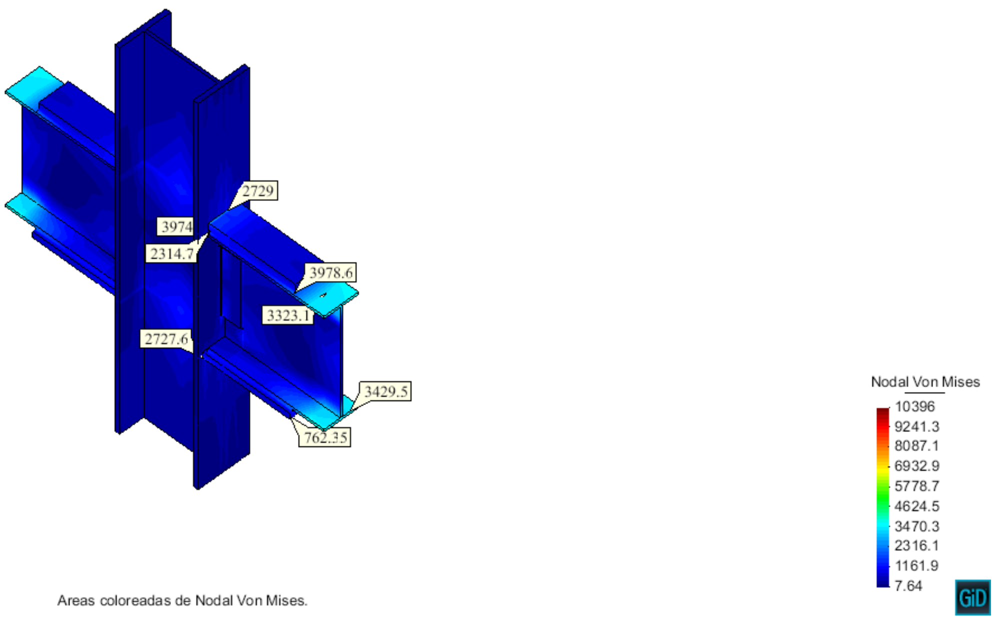


Figure 78: Shear stresses (YZ plane, SYZ) in M3-A36 conventional model.

Observations:

- Extremities of longitudinal fillet welds to resist bending moment reach a certain concentration on tangential stress below for their theoretical strength, and an amount up of 20% for restricted under ANSI-AISC 360-16 specifications.

Finite Element Analysis results: Model 3, Von Mises stresses VM_s.



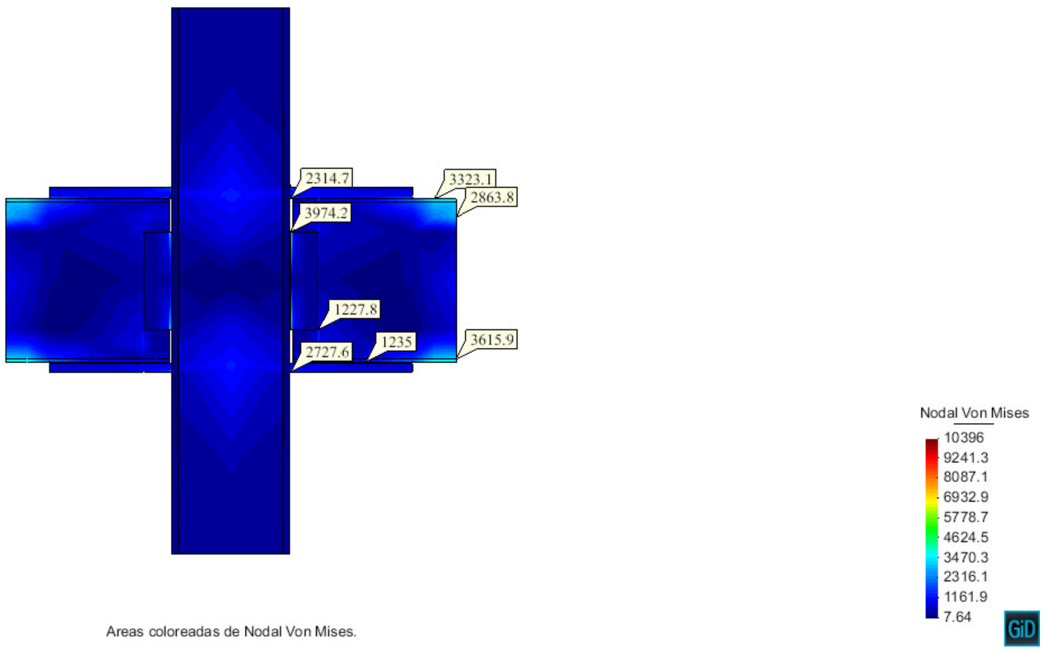


Figure 79: Von Mises stresses in M3-A36 conventional model.

Observations:

- Now, its seen how the flanges start to reach closely the yield stress for the free ends after bending plates seated.
- Atypical shapes start to emerge at the web of the column located between beams arms connected.

6.2 Non-qualified Bending Moment Configurations

6.2.1 Model 0.

Input Data

Name of model: M0-BM0-CL0-A36, (arbitrary model)
Beam: (BM_0) W12x30 — Column: (CL_0) W16x100 — Steel Designation: ASTM-A572 Gr. 50
F_y= 3515kg/cm^2 (50ksi) — F_u= 4570kg/cm^2 (65ksi) — Thickness governant: Column
The flange of beam fits in the flange of column ($0.85b_{fc} > b_{fb}$).
From beam:

- M_{pf}: 1.99 $e+06$ $kg \cdot cm$ flanges plastic moment contribution.
- M_{pw}: 0.49 $e+06$ $kg \cdot cm$ web plastic moment contribution.
- M_p: 2.48 $e+06$ $kg \cdot cm$ total plastic moment.
- M_y: 2.22 $e+06$ $kg \cdot cm$ yield moment.
- $\dfrac{M_{pf}}{M_p}$ = 80.35% percentage contribution of flanges respect to total plastic moment.
- $\dfrac{M_{pw}}{M_p}$ = 19.65% percentage contribution of web respect to total plastic moment.

Design forces:

- V_u = 1.29 $e+04$ kg ultimate shear load.
- V_s = 0.92 $e+04$ kg service shear load.
- M_u = 1.78 $e+06$ $kg \cdot cm$ ultimate moment.
- M_s = 1.27 $e+06$ $kg \cdot cm$ service moment.

Rigid Upper Plate

From beam:

Flexural strength.		
$F(kg \cdot cm)$	LRFD (ϕM_n)	ASD ($\frac{M_n}{\Omega}$)
Nominal moment	2.23e+06	1.49e+06

$F_{ut} = F_{uc}$ = 0.64 $e+05$ kg ultimate tension/compression force to design bending plates (according to M_u).
$F_{st} = F_{sc}$ = 0.41 $e+05$ kg service tension/compression force to design bending plates (according to M_s).

Resultant dimensions:

- b_p = 14.29cm (5.625in) width of plate.
- t_p = 2.54cm (1.00in) thickness of plate.
- L_p = 25.00cm (10.00in) length of plate.
- ψ = 1.12 shape factor.

Upper plate nominal force.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal yield force	0.82e+05	0.55e+05
Nominal rupture force	1.11e+05	0.74e+05

Fillet Welds for Rigid Plates

Results:

- $EXX/FX = 4900 kg/cm^2$ $(70 ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.80 cm$ minimum leg size of weld.
- $tw_{max} = 2.02 cm$ maximum leg size of weld.
- $w_p = 1.00 cm$ leg size of weld for design of project.
- $Lw_{req} = 40.99 cm$ length of weld required.
- $Lw_p = 48.00 cm$ length of weld to be adjusted to the project.
- $Lw_{eff} = 48.00 cm$ effective length of proportioned weld.
- $R_{nw} = 2078.58 kg/cm$ theoretical strength of weld.

Nominal strength of fillet weld.		
$F(kg/cm)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal strength	1558.94	1039.29

Nominal force due to fillet welds.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal force	0.75e+05	0.50e+05

Rigid Lower Plate

Results:

- $b_p = 21.59 cm$ $(8.50 in)$ width of plate.
- $t_p = 1.91 cm$ $(0.75 in)$ thickness of plate.
- $L_p = 25.00 cm$ $(10.00 in)$ length of plate.
- $r = 0.55 cm$ gyration radius of plate.
- $k = 0.65$ effective length factor.
- $kL_p = 16.25 cm$ effective length.
- $\frac{kL_p}{r} = 29.55$ slenderness ratio (adimensional).
- $F_e = 23.06\ e+03\ kg/cm^2$ Euler stress.
- $F_{cr} = 2.42\ e+03\ kg/cm^2$ critical stress.
- $P_n = 0.99\ e+05\ kg$ theoretical compression force resisted.

Compression force resisted by lower plate.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal force	0.89e+05	0.59e+05

Shear Design

Beam checking:

- $\frac{h}{t_w}$ = 47.31 slenderness ratio of web.
- k_v = 5.34 (a=0, no transverse stiffeners) adimensional value.
- C_{v1} = 1.00 adimensional value.
- $A_w = 20.63cm^2$ area of the web.

Shear strength of beam section.		
$F(kg)$	LRFD (ϕV_n)	ASD ($\frac{V_n}{\Omega}$)
Nominal shear resistance	3.92e+04	2.61e+04

Angles:

- Angle: $L3\ x\ 2\ x\ \frac{3}{8}$
- $b_p = 5.08cm$ $(2.00in)$ width of plate.
- $t_p = 0.95cm$ $(0.375in)$ thickness of plate.
- L_p or $h = 16.51cm$ $(6.50in)$ length of plate.

Strength of angles (the pair).		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal yield strength	4.30e+04	2.86e+04
Nominal rupture strength	5.77e+04	3.85e+04

Results for fillet welds of "back-to-back" angles:

- $EXX/FX = 4900kg/cm^2$ $(70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.50cm$ minimum leg size of weld.
- $tw_{max} = 0.75cm$ maximum leg size of weld.
- $w_p = 0.50cm$ leg size of weld for design of project.
- $Lw_p = 33.02cm$ length of weld proportioned.
- $Lw_{eff} = 33.02cm$ effective length of proportioned weld.

Nominal strength of fillet welds for angles.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal strength	2.06e+04	1.37e+04

Additional quantities.		
Weight	*Length of fillet weld (total)*	*Length of groove weld (total)*
18.12 Kg	66.04 cm of 5.0 mm, and 96.00 cm of 10 mm	35.88 cm

Flanges and Webs with concentrated forces (Chapter J.10)

Flange local bending.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	2.47e+04	1.64e+04
Column	12.37e+04	8.23e+04

From column:

- $t_w = 1.49cm$ thickness of web.
- $k = 3.53cm$ eccentricity of W shape.
- $l_b = 3.91cm$ bearing length (min(t_p,t_{fb})+$2w$); where:
  - w is the leg size of weld project (w_p).
  - t_p is the thicker thickness of the bending plates.
  - t_{fb} is the thickness of the flange beam.

Web local yielding.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	10.74e+04	7.16e+04

From column:

- $t_f = 2.50cm$ thickness of flange.
- $h = 43.18cm$ depth of W shape.
- $l_b = 3.91cm$ bearing length.
- $Q_f = 1.00$ adimensional factor.

Web local crippling.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	16.36e+04	10.91e+04

Transverse stiffeners (Chapter G2, Section 3 from ANSI/AISC-360-16):

Feature	**Beam**	**Column**
h	47.31cm	43.18cm
t_w	0.66cm	1.49cm
$\frac{h}{t_w}$	47.31	29.06
$2.46\sqrt{\frac{E}{F_{yw}}}$	59.26	59.26
Requirement?	NO	NO

- $F_{yw} = 3515kg/cm^2$ yield stress.
- Transverse stiffeners **not necessary if**:

$$\frac{h}{t_w} < 2.46\sqrt{\frac{E}{F_{yw}}}$$

Stiffening/Continuity Plates

- $C_u = T_u$: 0.00
- A_{req}: 0.00
- $\frac{b_{fb}}{3}$: $5.52cm$
- $0.56\sqrt{\frac{E}{F_y}}$: 15.90

For upper plate ($t_{up} = 2.22cm$ ($0.875in$)):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	23.47	34.37	10.56	5.34e+04	3.56e+04
Mid plate	11.73	34.37	5.28	2.67e+04	1.78e+04
Optimal	0.00	34.37	0.00	0.00	0.00

For lower plate ($t_{lp} = 1.91cm$ ($0.75in$)):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	20.12	34.37	10.56	4.58e+04	3.05e+04
Mid plate	10.06	34.37	5.28	2.29e+04	1.52e+04
Optimal	0.00	34.37	0.00	0.00	0.00

Note: strengths (kg) are expressed for just one plate.
For welds (two alternatives, it considers upper and lower edges):

- t_{w1} (weld leg size for alternative 1) $= 0.30cm$ $(\frac{1}{8})''$
- t_{w2} (weld leg size for alternative 2) $= 0.50cm$

Plate	$LRFD(\phi R_{n1})$	$ASD(\frac{R_{n1}}{\Omega})$	$LRFD(\phi R_{n2})$	$ASD(\frac{R_{n2}}{\Omega})$
FP/MP/OP	3.21e+04	2.14e+04	5.36e+04	3.57e+04

Observations

- Plates have a work performance up to 69% for yield (M_u), and about of 64-77% for ultimate design forces.
- Strength is delimited by resistance of welds.
- There is not need of stiffener elements.

Finite Element Analysis results: Model 0, Displacements.

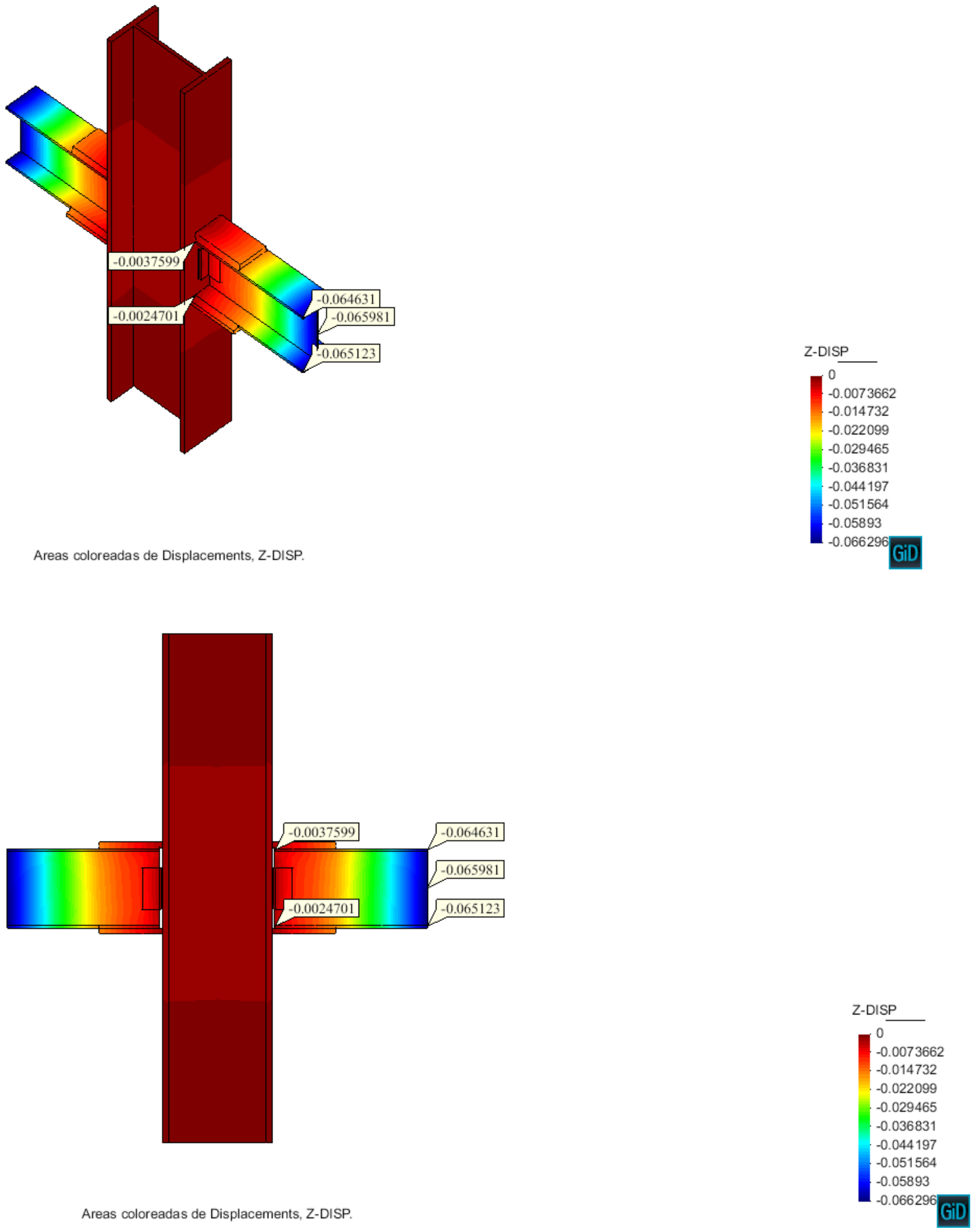
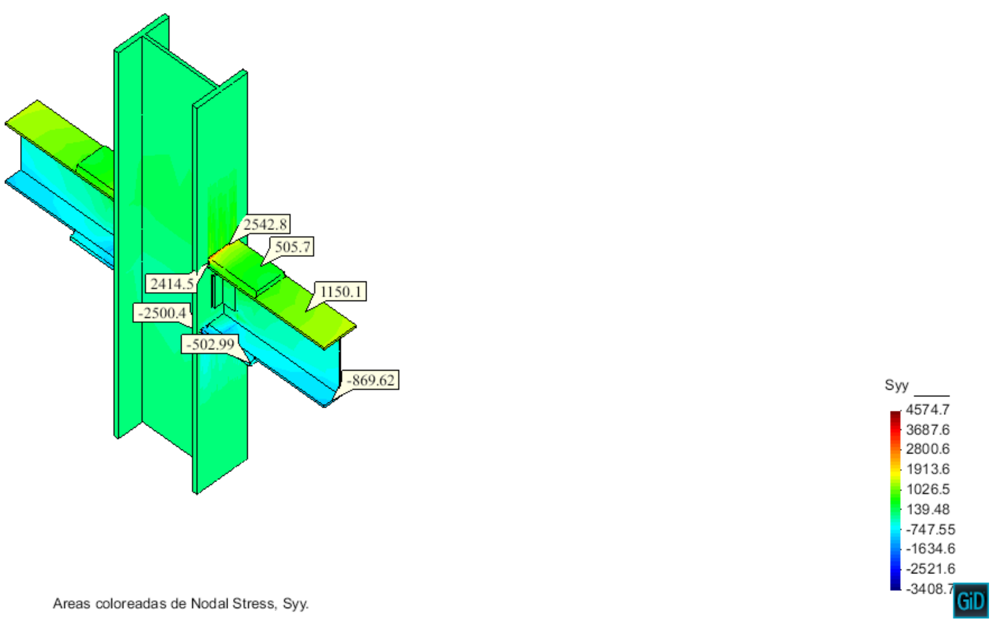


Figure 80: Vertical displacements (Z-axis) in M0-A36 N-QLF model.

Observations:

- Now, a difference respect to conventional model, the points located at free end get more deflected than those near from the column flange.

Finite Element Analysis results: Model 0, Tension stresses S_{yy}.



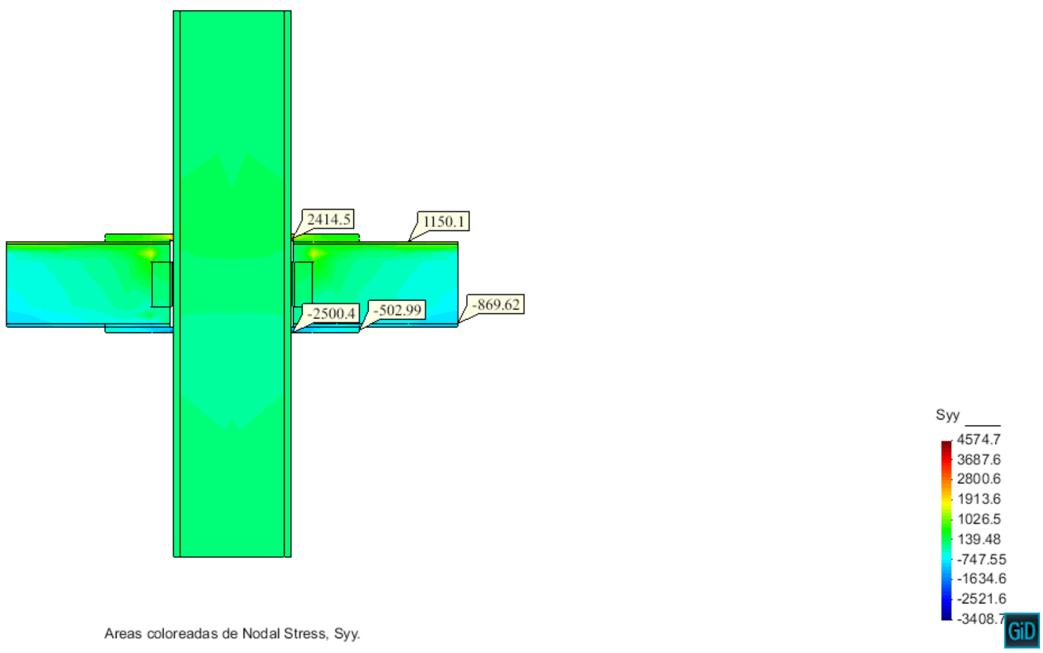


Figure 81: Axial stresses (Y-axis, SYY) in M0-A36 N-QLF model.

Observations:

- Axial stresses (y-direction) are close to limit yield value.
- By the coloured surfaces, the neutral axis of the beams seem displaced, having more fibers in compression.

Finite Element Analysis results: Model 0, Shear stresses S_{xy}.

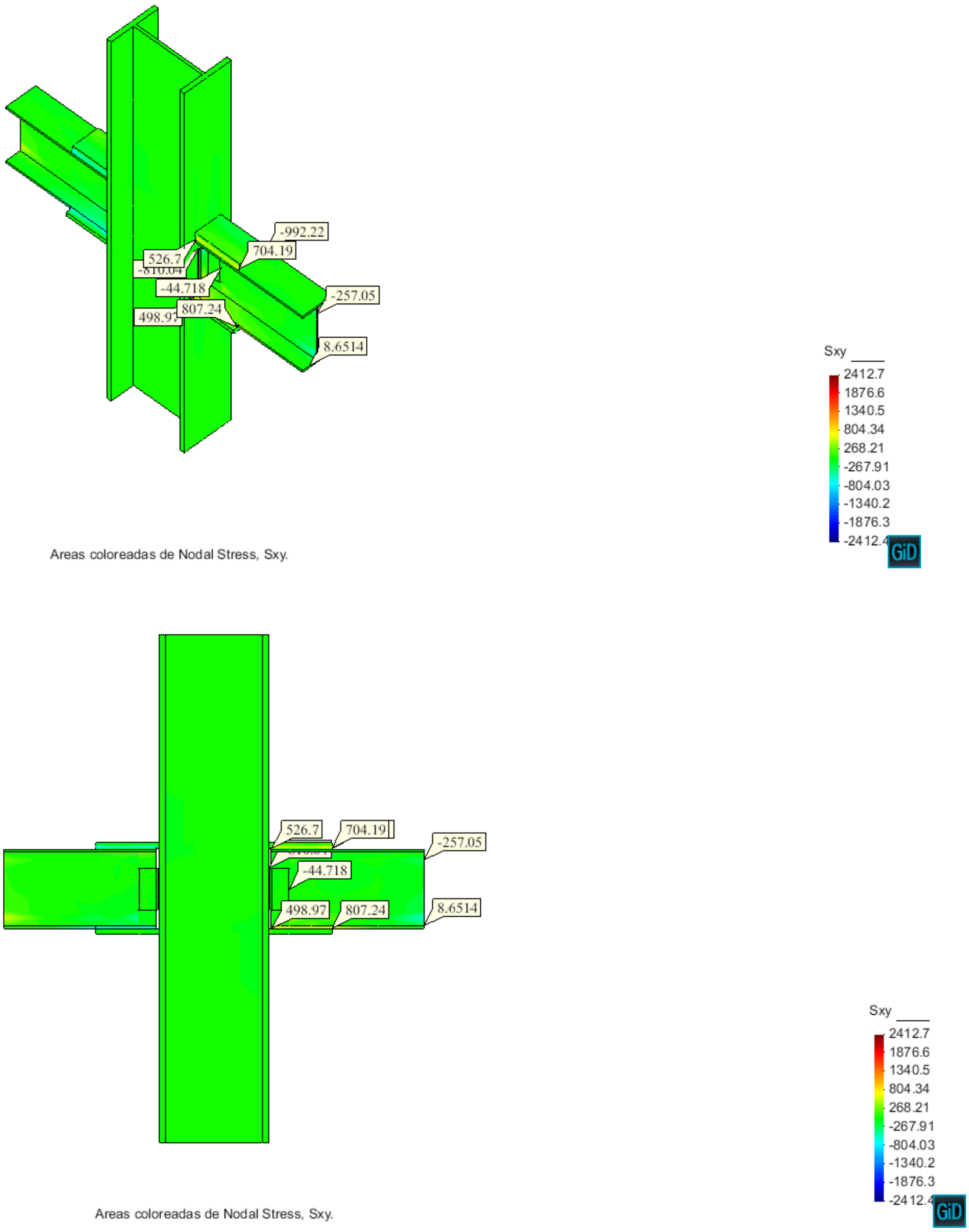


Figure 82: Shear stresses (XY plane, SXY) in M0-A36 N-QLF model.

Observations:

- All values are numbers below the restricted for nominal strength and theoretical resistance.

Finite Element Analysis results: Model 0, Shear stresses S_{yz}.

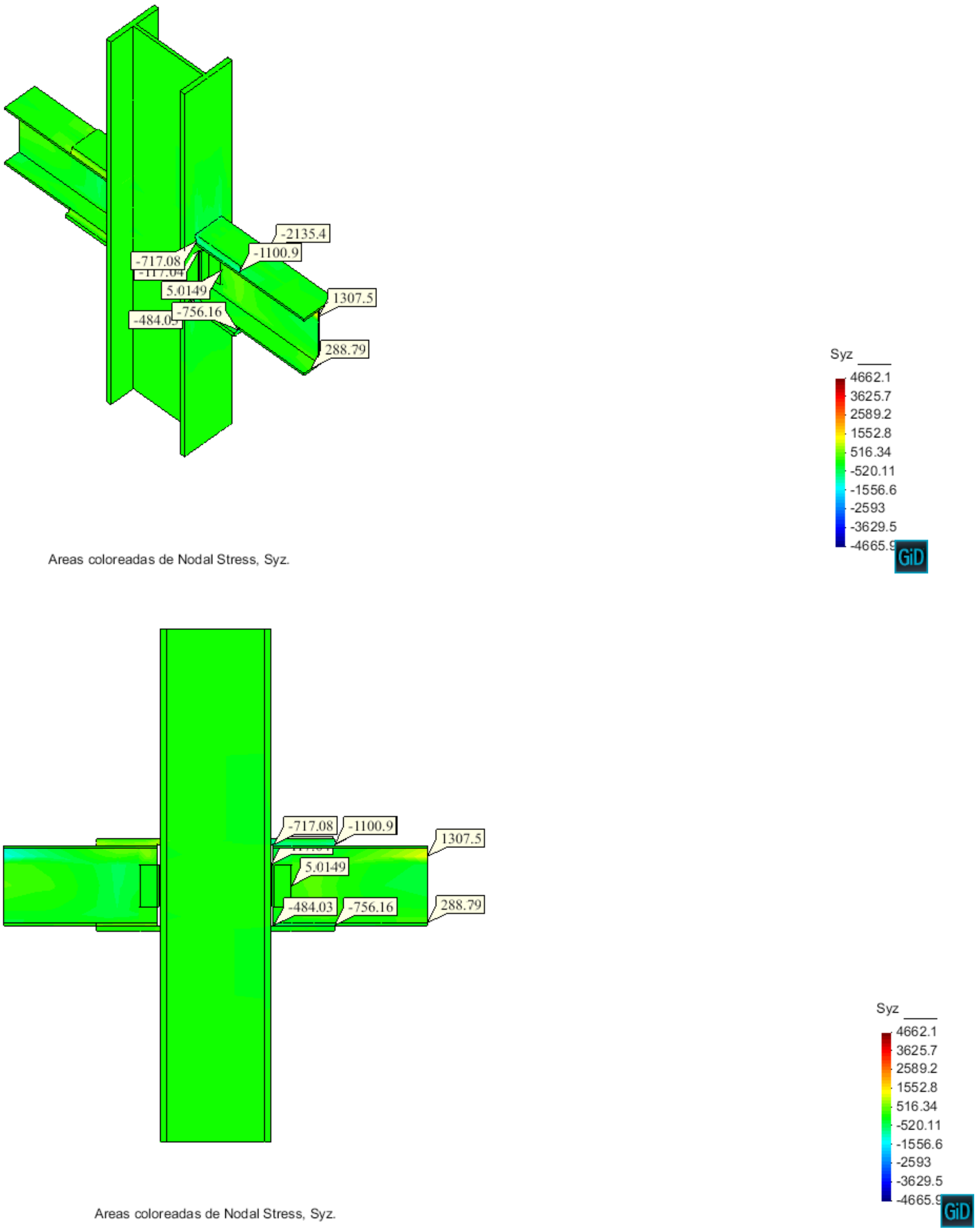


Figure 83: Shear stresses (YZ plane, SYZ) in M0-A36 N-QLF model.

Observations:

- The upper fillet welds have more concentration for the borders of extremities up to the theoretical strength.
- The shear stresses in the web-flange transition are less concentrated than the conventional model.

Finite Element Analysis results: Model 0, Von Mises stresses VM_s.

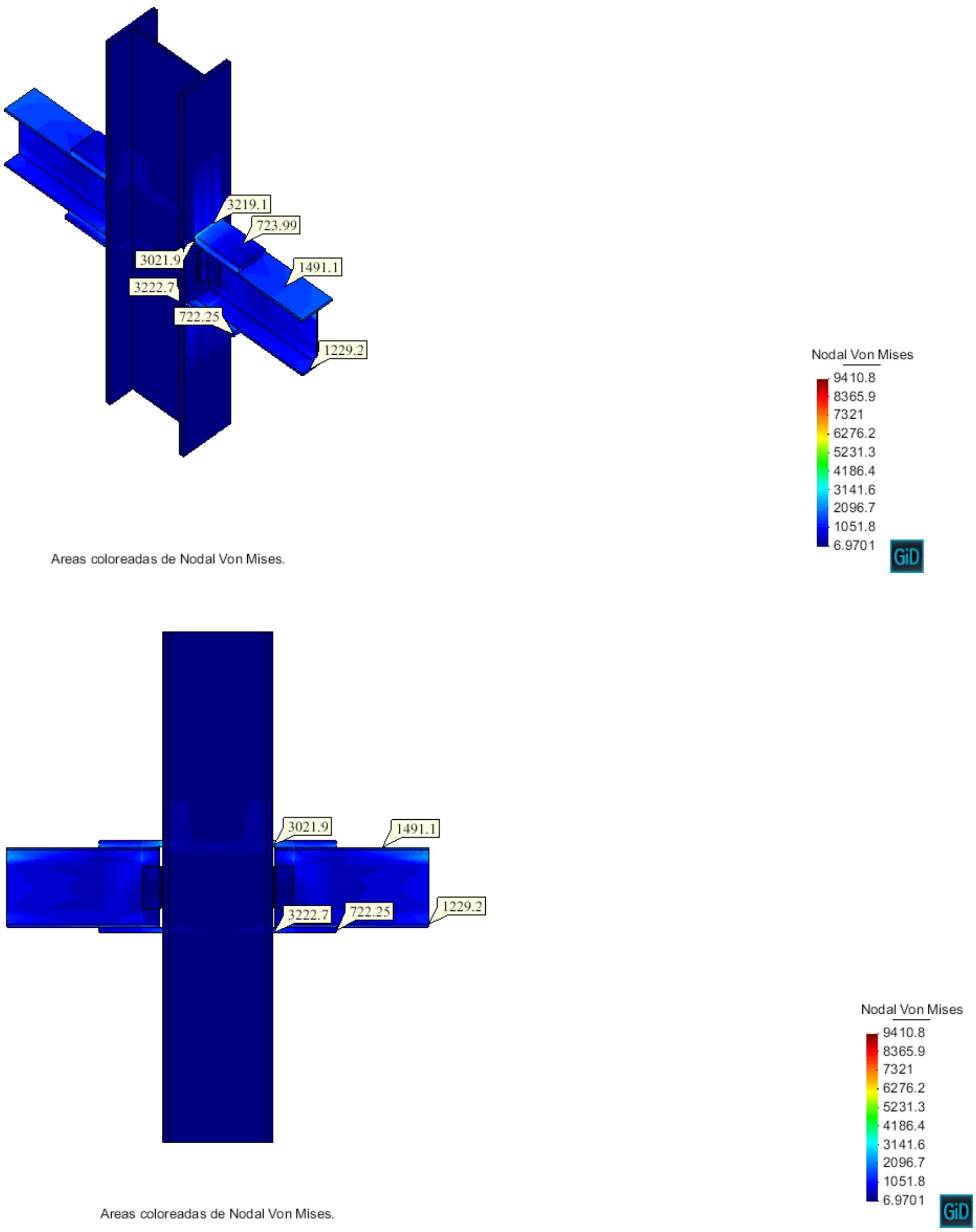


Figure 84: Von Mises stresses in M0-A36 N-QLF model.

Observations:

- The extreme edges of the bending plates with faces looking at shear plate have more concentration of stress with very similar values of the opposite faces.
- The wall of column flange connected have more concentration of stress than the conventional model.

6.2.2 Model 1.

Input Data

Name of model: M1-BM1-CL1-A36, (model for offices building)
Beam: (BM_1) W16x45 — Column: (CL_1) W16x100 — Steel Designation: ASTM-A572 Gr. 50
F_y= $3515 kg/cm^2$ $(50 ksi)$ — F_u= $4570 kg/cm^2$ $(65 ksi)$ — Thickness governant: Column
The flange of beam fits in the flange of column $(0.85 b_{fc} > b_{fb})$.
From beam:

- M_{pf}: $3.62\ e+06\ kg \cdot cm$ flanges plastic moment contribution.
- M_{pw}: $1.11\ e+06\ kg \cdot cm$ web plastic moment contribution.
- M_p: $4.74\ e+06\ kg \cdot cm$ total plastic moment.
- M_y: $4.19\ e+06\ kg \cdot cm$ yield moment.
- $\dfrac{M_{pf}}{M_p} = 76.50\%$ percentage contribution of flanges respect to total plastic moment.
- $\dfrac{M_{pw}}{M_p} = 23.50\%$ percentage contribution of web respect to total plastic moment.

Design forces:

- $V_u = 3.01\ e+04\ kg$ ultimate shear load.
- $V_s = 1.74\ e+04\ kg$ service shear load.
- $M_u = 3.81\ e+06\ kg \cdot cm$ ultimate moment.
- $M_s = 2.26\ e+06\ kg \cdot cm$ service moment.

Rigid Upper Plate

From beam:

Flexural strength.		
$F(kg \cdot cm)$	LRFD (ϕM_n)	ASD $(\frac{M_n}{\Omega})$
Nominal moment	4.26e+06	2.84e+06

$F_{ut} = F_{uc} = 0.89\ e+05\ kg$ ultimate tension/compression force to design bending plates (according to M_u).
$F_{st} = F_{sc} = 0.55\ e+05\ kg$ service tension/compression force to design bending plates (according to M_s).

Resultant dimensions:

- $b_p = 15.56 cm$ $(6.125 in)$ width of plate.
- $t_p = 3.18 cm$ $(1.25 in)$ thickness of plate.
- $L_p = 35.00 cm$ $(14.00 in)$ length of plate.
- $\psi = 1.13$ shape factor.

Upper plate nominal force.		
$F(kg)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal yield force	1.12e+05	0.75e+05
Nominal rupture force	1.51e+05	1.00e+05

Fillet Welds for Rigid Plates

Results:

- $EXX/FX = 4900kg/cm^2$ $(70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.80cm$ minimum leg size of weld.
- $tw_{max} = 2.98cm$ maximum leg size of weld.
- $w_p = 1.00cm$ leg size of weld for design of project.
- $Lw_{req} = 56.85cm$ length of weld required.
- $Lw_p = 68.00cm$ length of weld to be adjusted to the project.
- $Lw_{eff} = 68.00cm$ effective length of proportioned weld.
- $R_{nw} = 2078.58kg/cm$ theoretical strength of weld.

Nominal strength of fillet weld.		
$F(kg/cm)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal strength	1558.94	1039.29

Nominal force due to fillet welds.		
$F(kg)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal force	1.06e+05	0.71e+05

Rigid Lower Plate

Results:

- $b_p = 21.59cm$ $(8.50in)$ width of plate.
- $t_p = 2.22cm$ $(0.875in)$ thickness of plate.
- $L_p = 35.00cm$ $(14.00in)$ length of plate.
- $r = 0.642cm$ gyration radius of plate.
- $k = 0.65$ effective length factor.
- $kL_p = 22.75cm$ effective length.
- $\frac{kL_p}{r} = 35.46$ slenderness ratio (adimensional).
- $F_e = 16.01\ e+03\ kg/cm^2$ Euler stress.
- $F_{cr} = 2.37\ e+03\ kg/cm^2$ critical stress.
- $P_n = 1.13\ e+05\ kg$ theoretical compression force resisted.

Compression force resisted by lower plate.		
$F(kg)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal force	1.02e+05	0.68e+05

Shear Design

Beam checking:

- $\frac{h}{t_w} = 46.67$ slenderness ratio of web.
- $k_v = 5.34$ (a=0, no transverse stiffeners) adimensional value.
- $C_{v1} = 1.00$ adimensional value.
- $A_w = 35.84cm^2$ area of the web.

Shear strength of beam section.		
$F(kg)$	LRFD (ϕV_n)	ASD ($\frac{V_n}{\Omega}$)
Nominal shear resistance	6.80e+04	4.53e+04

Angles:

- Angle: $L3\ x\ 2\frac{1}{2}\ x\ \frac{3}{8}$
- $b_p = 7.62cm\ (3.00in)$ width of plate.
- $t_p = 0.95cm\ (0.375in)$ thickness of plate.
- $L_p\ or\ h = 25.40cm\ (10.00in)$ length of plate.

Strength of angles (the pair).		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal yield strength	6.61e+04	4.40e+04
Nominal rupture strength	8.88e+04	5.92e+04

Results for fillet welds of "back-to-back" angles:

- $EXX/FX = 4900kg/cm^2\ (70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.50cm$ minimum leg size of weld.
- $tw_{max} = 0.75cm$ maximum leg size of weld.
- $w_p = 0.50cm$ leg size of weld for design of project.
- $Lw_p = 50.80cm$ length of weld proportioned.
- $Lw_{eff} = 50.80cm$ effective length of proportioned weld.

Nominal strength of fillet welds for angles.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal strength	3.17e+04	2.11e+04

Additional quantities.		
Weight	*Length of fillet weld (total)*	*Length of groove weld (total)*
31.74 Kg	101.6 cm of 5.0 mm, and 136.00 cm of 10 mm	37.15 cm

Flanges and Webs with concentrated forces (Chapter J.10)

Flange local bending.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	4.07e+04	2.71e+04
Column	12.37e+04	8.23e+04

From column:

- $t_w = 1.49cm$ thickness of web.
- $k = 3.53cm$ eccentricity of W shape.
- $l_b = 4.22cm$ bearing length (min(t_p,t_{fb})+2w); where:
  - w is the leg size of weld project (w_p).
  - t_p is the thicker thickness of the bending plates.
  - t_{fb} is the thickness of the flange beam.

Web local yielding.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	10.90e+04	7.27e+04

From column:

- $t_f = 2.50cm$ thickness of flange.
- $h = 43.18cm$ depth of W shape.
- $l_b = 4.22cm$ bearing length.
- $Q_f = 1.00$ adimensional factor.

Web local crippling.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	16.51e+04	11.00e+04

Transverse stiffeners (Chapter G2, Section 3 from ANSI/AISC-360-16):

Feature	**Beam**	**Column**
h	40.89cm	43.18cm
t_w	0.88cm	1.49cm
$\frac{h}{t_w}$	46.67	29.06
$2.46\sqrt{\frac{E}{F_{yw}}}$	59.26	59.26
Requirement?	NO	NO

- $F_{yw} = 3515kg/cm^2$ yield stress.
- Transverse stiffeners **not necessary if**:

$$\frac{h}{t_w} < 2.46\sqrt{\frac{E}{F_{yw}}}$$

Stiffening/Continuity Plates

- $C_u = T_u$: 0.00
- A_{req}: $2.99cm^2$
- $\frac{b_{fb}}{3}$: $5.96cm$
- $0.56\sqrt{\frac{E}{F_y}}$: 15.90

For upper plate ($t_{up} = 3.18cm$ $(1.25in)$):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	35.54	34.37	10.56	8.09e+04	5.38e+04
Mid plate	17.77	34.37	5.28	4.04e+04	2.69e+04
Optimal	0.00	34.37	0.00	0.00	0.00

For lower plate ($t_{lp} = 2.22cm$ $(0.875in)$):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	24.88	34.37	10.56	5.66e+04	3.78e+04
Mid plate	12.44	34.37	5.28	2.83e+04	1.89e+04
Optimal	0.00	34.37	0.00	0.00	0.00

Note: strengths (kg) are expressed for just one plate.
For welds (two alternatives, it considers upper and lower edges):

- t_{w1} (weld leg size for alternative 1) = $0.30cm$ $(\frac{1}{8})''$
- t_{w2} (weld leg size for alternative 2) = $0.50cm$

Plate	$LRFD(\phi R_{n1})$	$ASD(\frac{R_{n1}}{\Omega})$	$LRFD(\phi R_{n2})$	$ASD(\frac{R_{n2}}{\Omega})$
FP/MP/OP	3.21e+04	2.14e+04	5.36e+04	3.57e+04

Observations

- Plates have a work performance up to 83% for yield (M_u), and about of 78% for ultimate design forces. The lower plate reaches failure by yielding first than the upper one, and later fillet welds do.
- There is not need of stiffener elements.

Finite Element Analysis results: Model 1, Displacements.

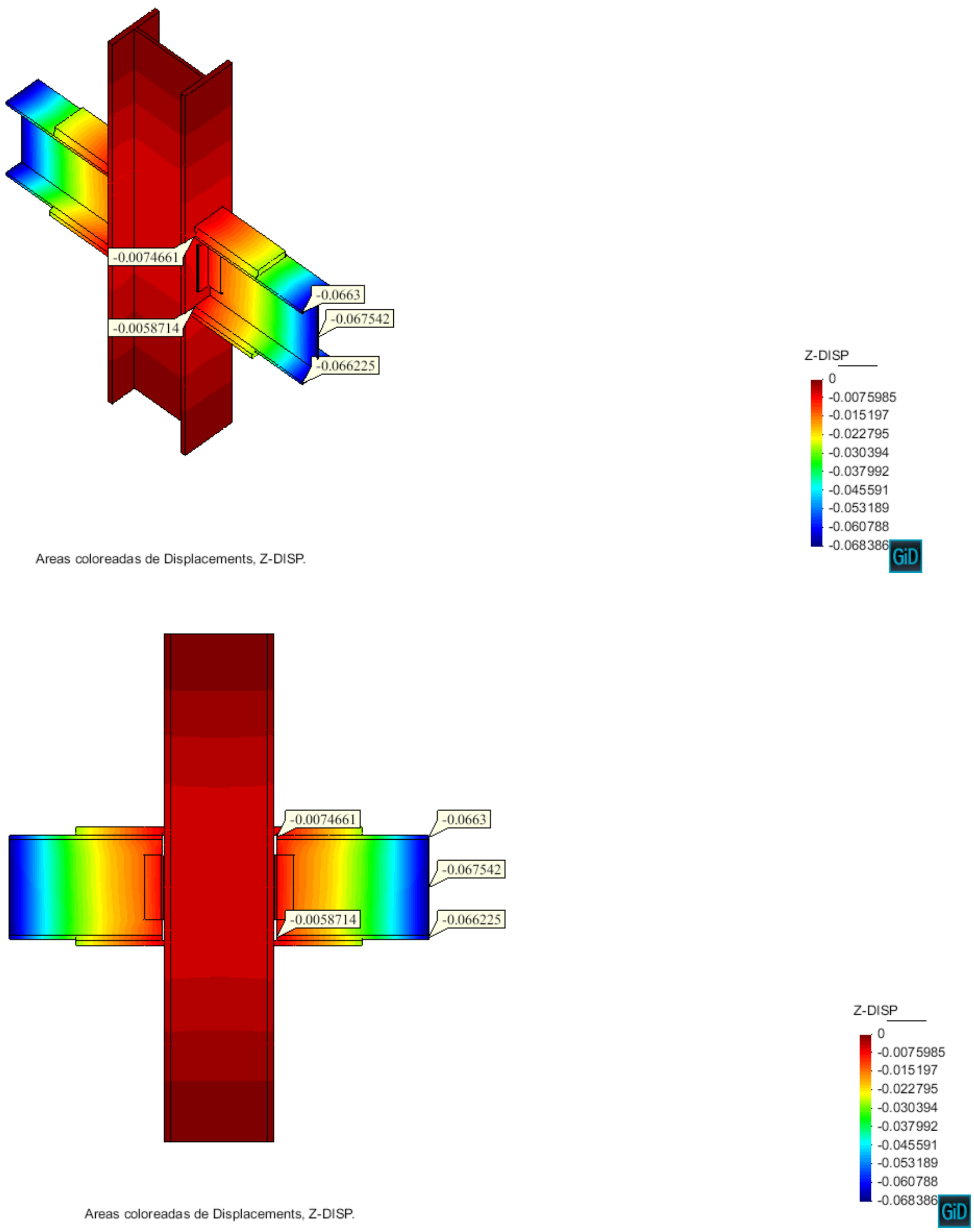


Figure 85: Vertical displacements (Z-axis) in M1-A36 N-QLF model.

Observations:

- This model have vertical displacements very similar to conventional models.

Finite Element Analysis results: Model 1, Tension stresses S_{yy}.

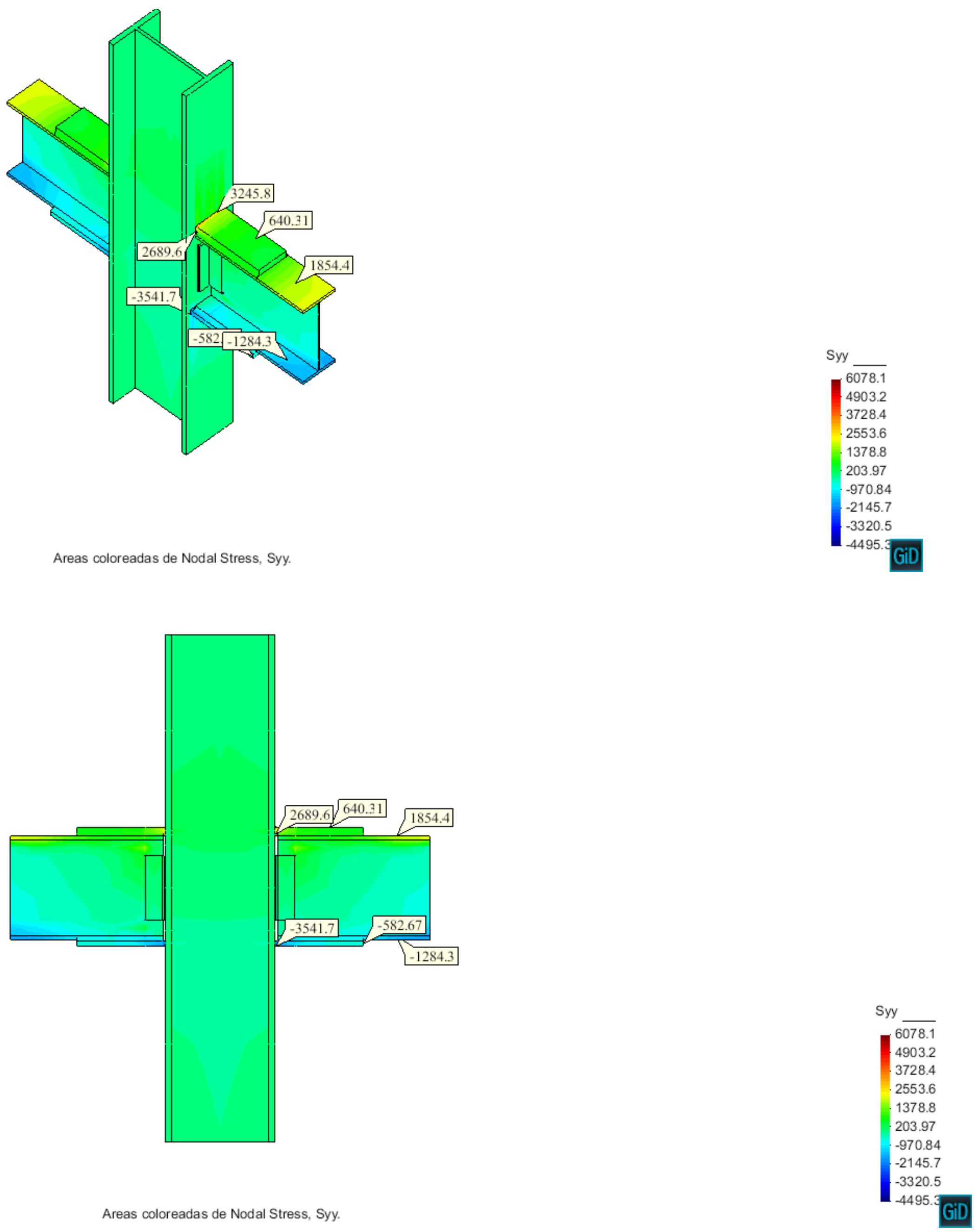
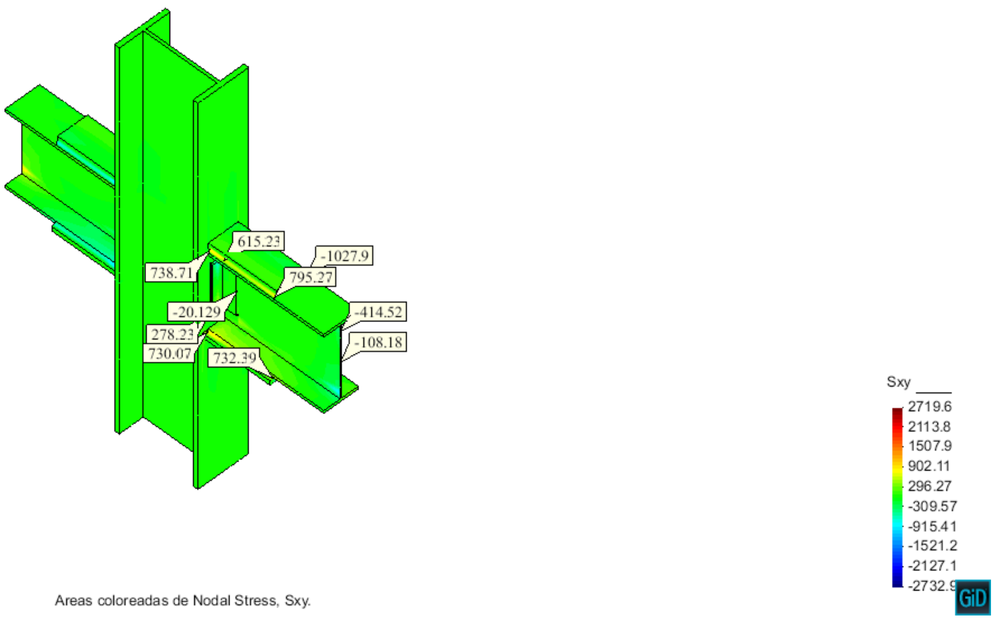


Figure 86: Axial stresses (Y-axis, SYY) in M1-A36 N-QLF model.

Observations:

- Tensile stresses are above yield stress (30%), equally as the correspondent conventional model.
- The extreme edges of the bending plates with faces looking at shear plate have more concentration of stress with very similar values of the opposite faces.

Finite Element Analysis results: Model 1, Shear stresses S_{xy}.



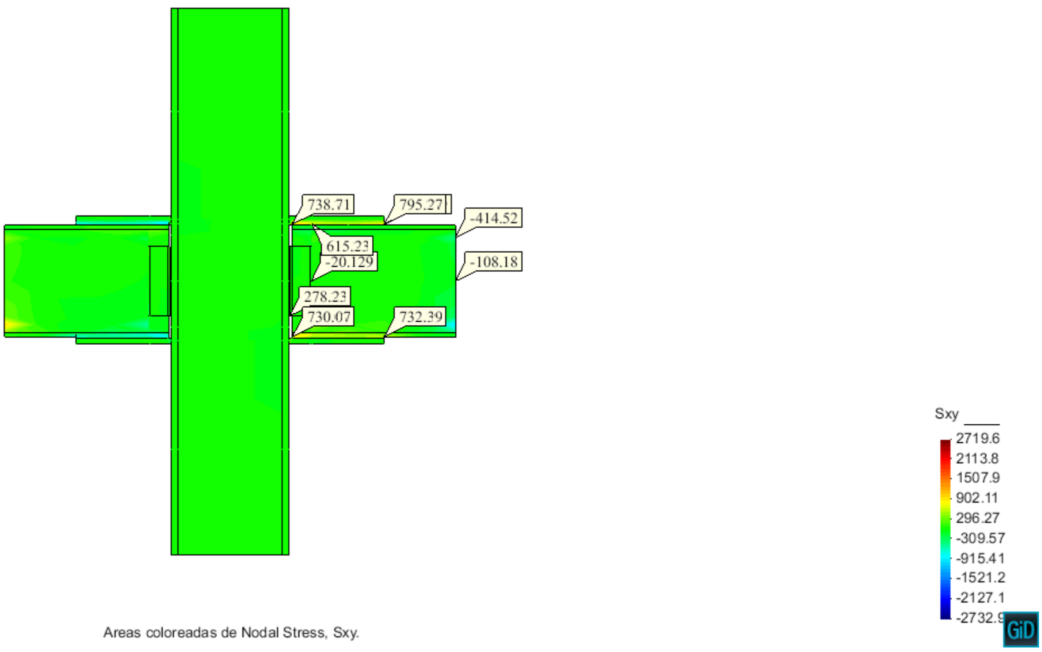
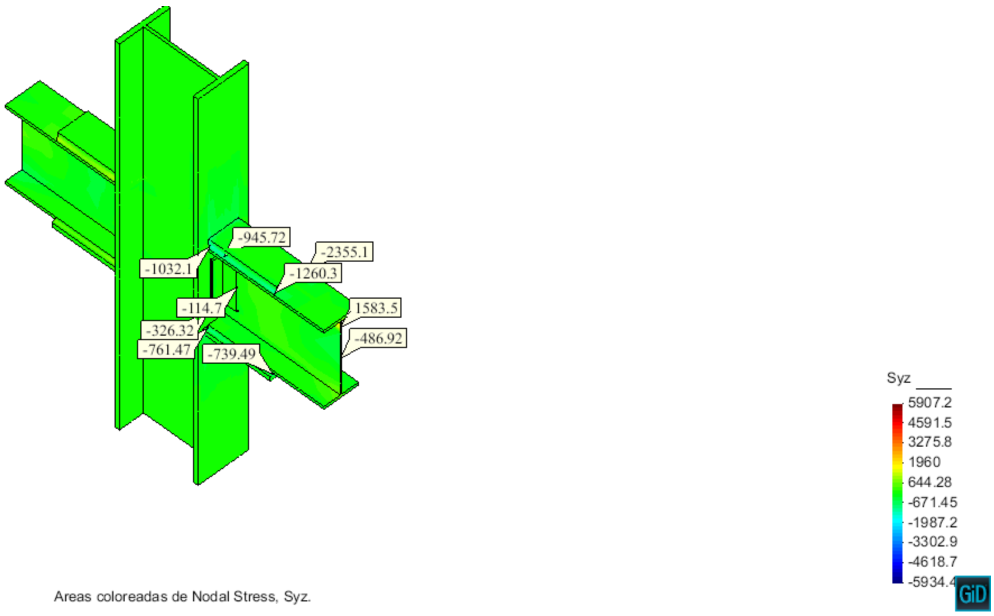


Figure 87: Shear stresses (XY plane, SXY) in M1-A36 N-QLF model.

Observations:

- The figures shown are seated below design shear forces and the theoretical ones.

Finite Element Analysis results: Model 1, Shear stresses S_{yz}.



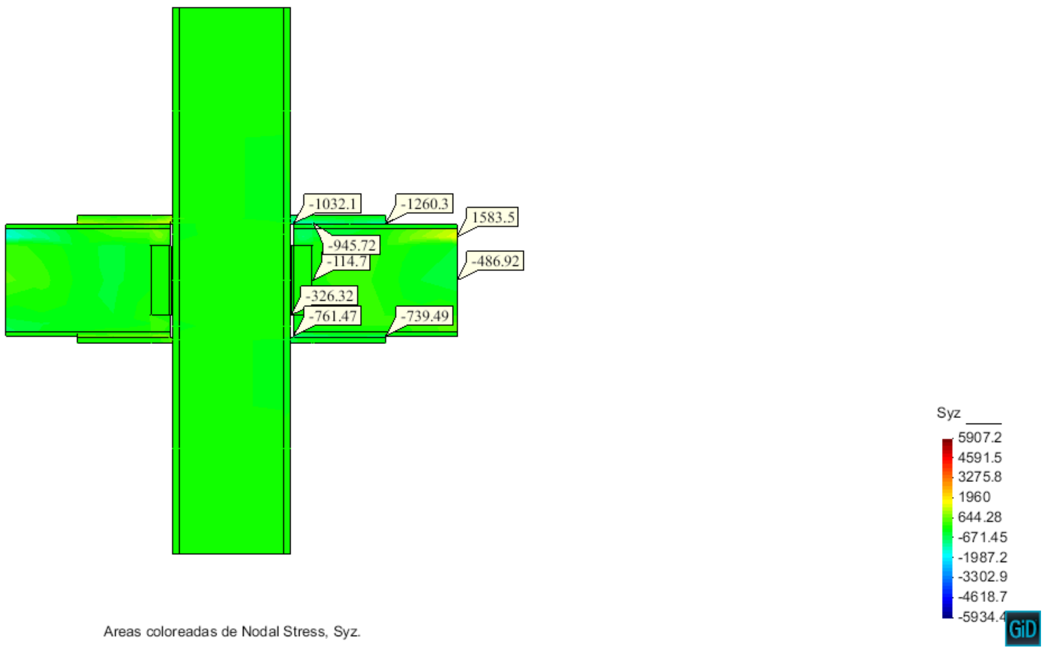


Figure 88: Shear stresses (YZ plane, SYZ) in M1-A36 N-QLF model.

Observations:

- Just those values located at the extreme edges of the gaps between beam and upper bending plate show results up to design stresses (theoretical and limit according to specifications employed).

Finite Element Analysis results: Model 1, Von Mises stresses VM_s.

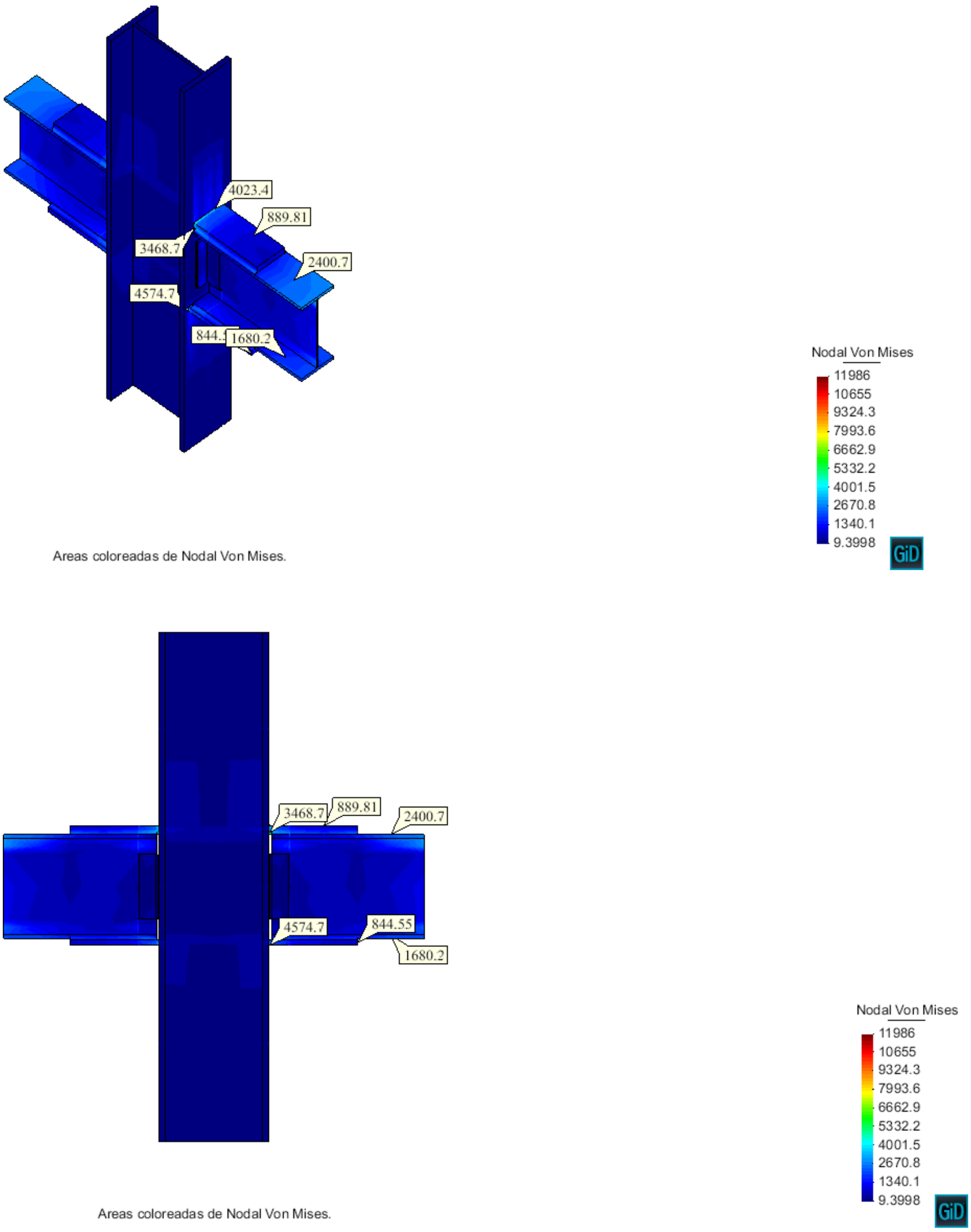


Figure 89: Von Mises stresses in M1-A36 N-QLF model.

Observations:

- Yield surfaces in the column web are more symmetrical than previous studied model, besides, the pattern seems to extend more to the flanges of the column and affecting less to the column web compared to conventional configuration.

6.2.3 Model 2.

Input Data

Name of model: M2-BM2-CL2-A36, (model for library building)
Beam: (BM_2) W18x46 — Column: (CL_2) W16x100 — Steel Designation: ASTM-A572 Gr. 50
F_y= $3515kg/cm^2$ ($50ksi$) — F_u= $4570kg/cm^2$ ($65ksi$) — Thickness governant: Column
The flange of beam fits in the flange of column ($0.85b_{fc} > b_{fb}$).
From beam:

- M_{pf}: $3.76\ e+06\ kg \cdot cm$ flanges plastic moment contribution.
- M_{pw}: $1.48\ e+06\ kg \cdot cm$ web plastic moment contribution.
- M_p: $5.24\ e+06\ kg \cdot cm$ total plastic moment.
- M_y: $4.54\ e+06\ kg \cdot cm$ yield moment.
- $\dfrac{M_{pf}}{M_p} = 71.76\%$ percentage contribution of flanges respect to total plastic moment.
- $\dfrac{M_{pw}}{M_p} = 28.24\%$ percentage contribution of web respect to total plastic moment.

Design forces:

- $V_u = 2.99\ e+04\ kg$ ultimate shear load.
- $V_s = 2.30\ e+04\ kg$ service shear load.
- $M_u = 3.34\ e+06\ kg \cdot cm$ ultimate moment.
- $M_s = 2.57\ e+06\ kg \cdot cm$ service moment.

Rigid Upper Plate

From beam:

Flexural strength.		
$F(kg \cdot cm)$	LRFD (ϕM_n)	ASD ($\frac{M_n}{\Omega}$)
Nominal moment	4.71e+06	3.13e+06

$F_{ut} = F_{uc} = 0.85\ e+05\ kg$ ultimate tension/compression force to design bending plates (according to M_u).
$F_{st} = F_{sc} = 0.56\ e+05\ kg$ service tension/compression force to design bending plates (according to M_s).

Resultant dimensions:

- $b_p = 13.02cm$ ($5.125in$) width of plate.
- $t_p = 3.81cm$ ($1.50in$) thickness of plate.
- $L_p = 35.00cm$ ($14.00in$) length of plate.
- $\psi = 1.15$ shape factor.

Upper plate nominal force.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal yield force	1.13e+05	0.75e+05
Nominal rupture force	1.51e+05	1.01e+05

Fillet Welds for Rigid Plates

Results:

- $EXX/FX = 4900kg/cm^2$ $(70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.80cm$ minimum leg size of weld.
- $tw_{max} = 3.61cm$ maximum leg size of weld.
- $w_p = 1.00cm$ leg size of weld for design of project.
- $Lw_{req} = 54.81cm$ length of weld required.
- $Lw_p = 68.00cm$ length of weld to be adjusted to the project.
- $Lw_{eff} = 68.00cm$ effective length of proportioned weld.
- $R_{nw} = 2078.58kg/cm$ theoretical strength of weld.

Nominal strength of fillet weld.		
$F(kg/cm)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal strength	1558.94	1039.29

Nominal force due to fillet welds.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal force	1.06e+05	0.71e+05

Rigid Lower Plate

Results:

- $b_p = 17.78cm$ $(7.00in)$ width of plate.
- $t_p = 2.54cm$ $(1.00in)$ thickness of plate.
- $L_p = 35.00cm$ $(14.00in)$ length of plate.
- $r = 0.733cm$ gyration radius of plate.
- $k = 0.65$ effective length factor.
- $kL_p = 22.75cm$ effective length.
- $\frac{kL_p}{r} = 31.03$ slenderness ratio (adimensional).
- $F_e = 20.91\ e+03\ kg/cm^2$ Euler stress.
- $F_{cr} = 2.40\ e+03\ kg/cm^2$ critical stress.
- $P_n = 1.09\ e+05\ kg$ theoretical compression force resisted.

Compression force resisted by lower plate.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal force	0.98e+05	0.65e+05

Shear Design

Beam checking:

- $\frac{h}{t_w} = 50.28$ slenderness ratio of web.
- $k_v = 5.34$ (a=0, no transverse stiffeners) adimensional value.
- $C_{v1} = 1.00$ adimensional value.
- $A_w = 42.04cm^2$ area of the web.

Shear strength of beam section.		
$F(kg)$	LRFD (ϕV_n)	ASD ($\frac{V_n}{\Omega}$)
Nominal shear resistance	7.98e+04	5.31e+04

Angles:

- Angle: $L3\ x\ 2\ x\ \frac{3}{8}$
- $b_p = 5.08cm\ (2.00in)$ width of plate.
- $t_p = 0.95cm\ (0.375in)$ thickness of plate.
- $L_p\ or\ h = 25.40cm\ (10.00in)$ length of plate.

Strength of angles (the pair).		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal yield strength	6.61e+04	4.40e+04
Nominal rupture strength	8.88e+04	5.92e+04

Results for fillet welds of "back-to-back" angles:

- $EXX/FX = 4900kg/cm^2\ (70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.50cm$ minimum leg size of weld.
- $tw_{max} = 0.75cm$ maximum leg size of weld.
- $w_p = 0.50cm$ leg size of weld for design of project.
- $Lw_p = 50.80cm$ length of weld proportioned.
- $Lw_{eff} = 50.80cm$ effective length of proportioned weld.

Nominal strength of fillet welds for angles.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal strength	3.17e+04	2.11e+04

Additional quantities.		
Weight	*Length of fillet weld (total)*	*Length of groove weld (total)*
30.54 Kg	101.6 cm of 5.0 mm, and 136.00 cm of 10 mm	30.80 cm

Flanges and Webs with concentrated forces (Chapter J.10)

Flange local bending.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	4.67e+04	3.11e+04
Column	12.37e+04	8.23e+04

From column:

- $t_w = 1.49cm$ thickness of web.
- $k = 3.53cm$ eccentricity of W shape.
- $l_b = 4.50cm$ bearing length (min(t_p,t_{fb})+2w); where:
  - w is the leg size of weld project (w_p).
  - t_p is the thicker thickness of the bending plates.
  - t_{fb} is the thickness of the flange beam.

Web local yielding.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	11.05e+04	7.37e+04

From column:

- $t_f = 2.50cm$ thickness of flange.
- $h = 43.18cm$ depth of W shape.
- $l_b = 4.50cm$ bearing length.
- $Q_f = 1.00$ adimensional factor.

Web local crippling.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	16.64e+04	11.09e+04

Transverse stiffeners (Chapter G2, Section 3 from ANSI/AISC-360-16):

Feature	**Beam**	**Column**
h	45.97cm	43.18cm
t_w	0.91cm	1.49cm
$\frac{h}{t_w}$	50.28	29.06
$2.46\sqrt{\frac{E}{F_{yw}}}$	59.26	59.26
Requirement?	NO	NO

- $F_{yw} = 3515kg/cm^2$ yield stress.
- Transverse stiffeners **not necessary if**:

$$\frac{h}{t_w} < 2.46\sqrt{\frac{E}{F_{yw}}}$$

Stiffening/Continuity Plates

- $C_u = T_u$: 0.00
- A_{req}: 1.51
- $\frac{b_{fb}}{3}$: 5.13cm
- $0.56\sqrt{\frac{E}{F_y}}$: 15.90

For upper plate ($t_{up} = 3.81cm\ (1.50in)$):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	42.65	34.37	10.56	9.71e+04	6.46e+04
Mid plate	21.32	34.37	5.28	4.85e+04	3.23e+04
Optimal	0.00	34.37	0.00	0.00	0.00

For lower plate ($t_{lp} = 2.54cm\ (1.00in)$):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	28.44	34.37	10.56	6.47e+04	4.31e+04
Mid plate	14.22	34.37	5.28	3.23e+04	2.15e+04
Optimal	0.00	34.37	0.00	0.00	0.00

Note: strengths (kg) are expressed for just one plate.
For welds (two alternatives, it considers upper and lower edges):

- t_{w1} (weld leg size for alternative 1) $= 0.30cm\ (\frac{1}{8})''$
- t_{w2} (weld leg size for alternative 2) $= 0.50cm$

Plate	$LRFD(\phi R_{n1})$	$ASD(\frac{R_{n1}}{\Omega})$	$LRFD(\phi R_{n2})$	$ASD(\frac{R_{n2}}{\Omega})$
FP/MP/OP	3.21e+04	2.14e+04	5.36e+04	3.57e+04

Observations

- Plates have a work performance up to 64% for yield (M_u), and about of 76-79% for ultimate design forces.
- Strength is delimited by yield of lower plate, later, fillet welds will reach failure later.
- There is not need of stiffener elements. The area required for continuity plates is almost negligible.

Finite Element Analysis results: Model 2, Displacements.

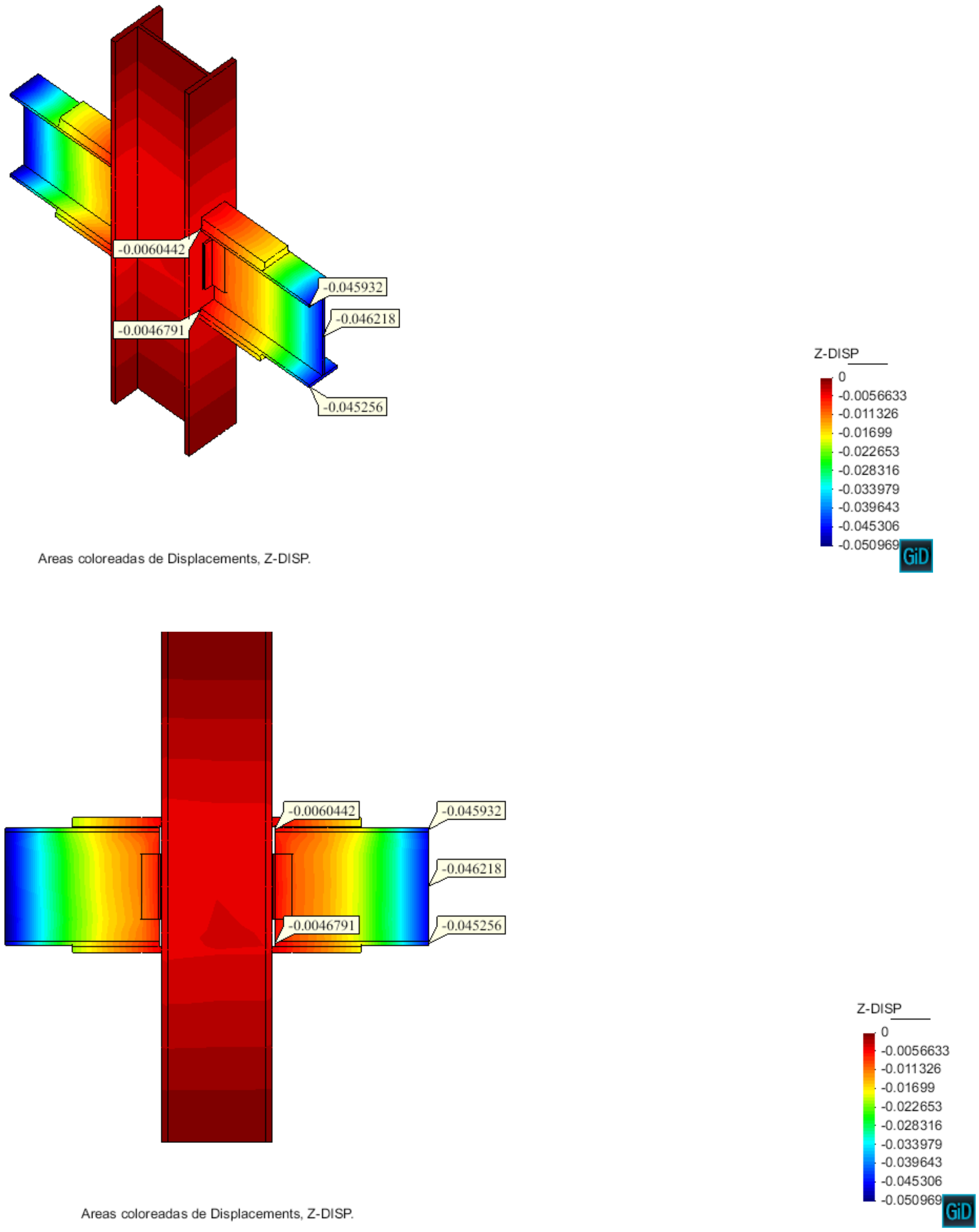


Figure 90: Vertical displacements (Z-axis) in M2-A36 N-QLF model.

Finite Element Analysis results: Model 2, Tension stresses S_{yy}.

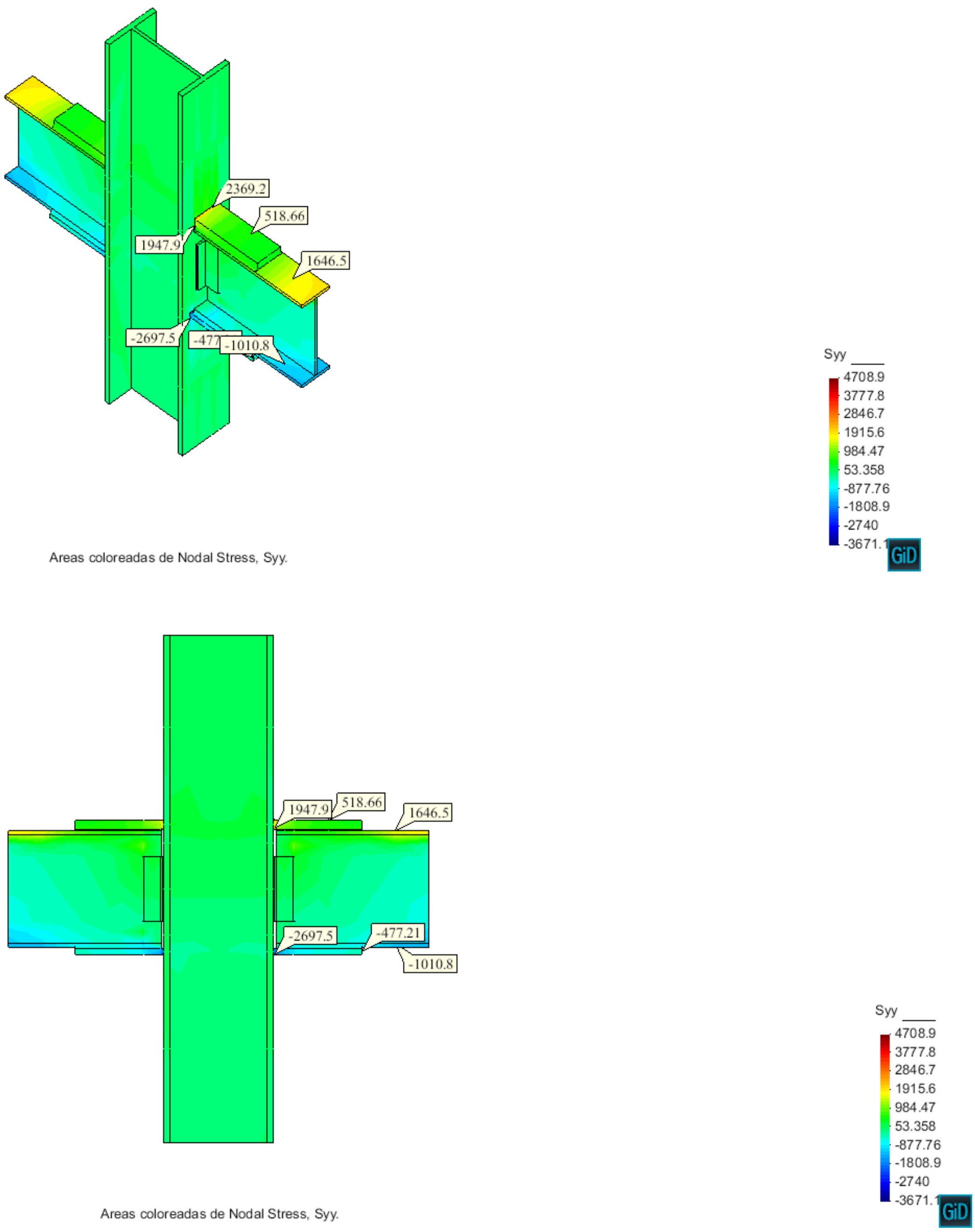


Figure 91: Axial stresses (Y-axis, SYY) in M2-A36 N-QLF model.

Observations:

- From the picture, it is seen that just the lower plates reach the yield stress parameter.

Finite Element Analysis results: Model 2, Shear stresses S_{xy}.

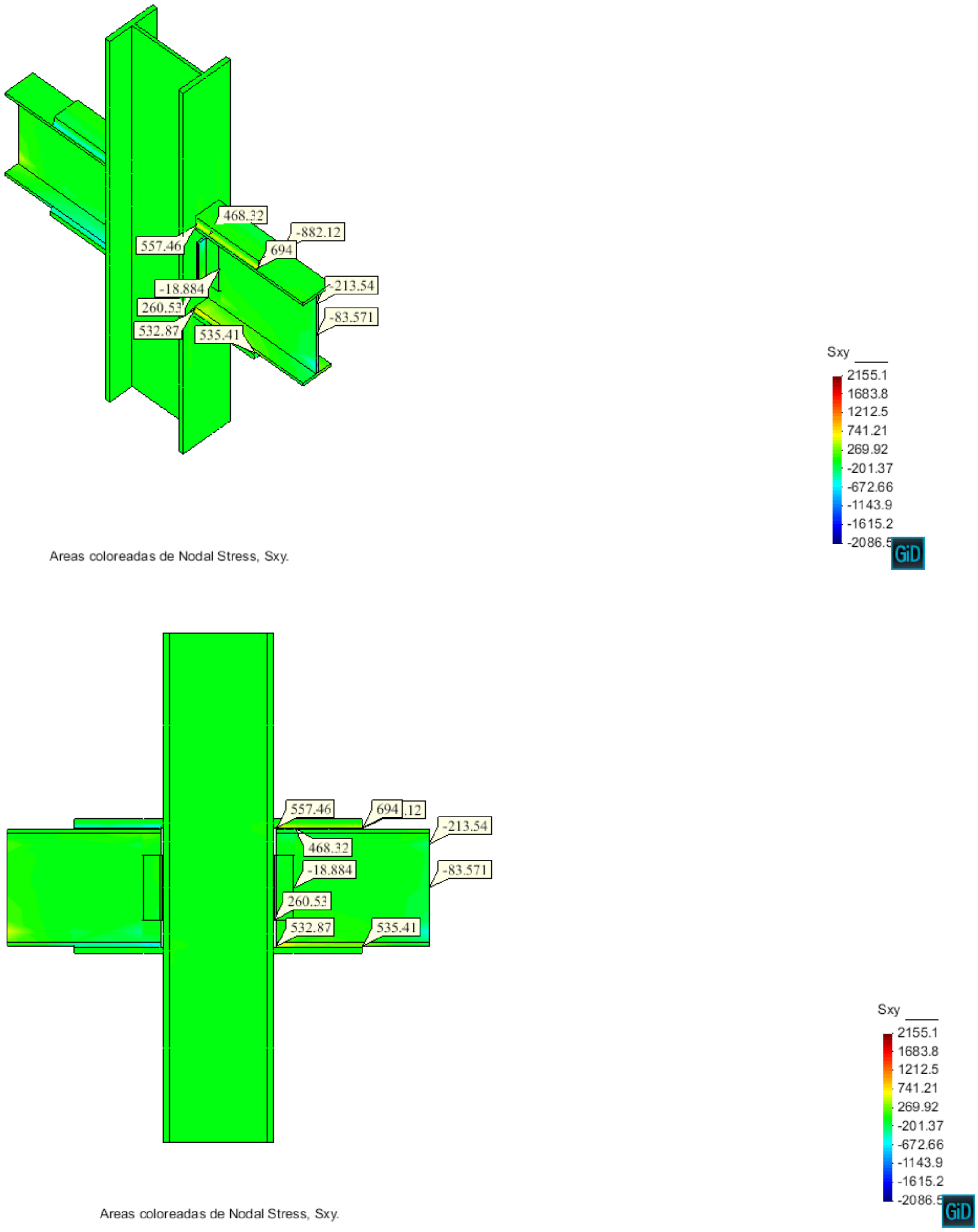
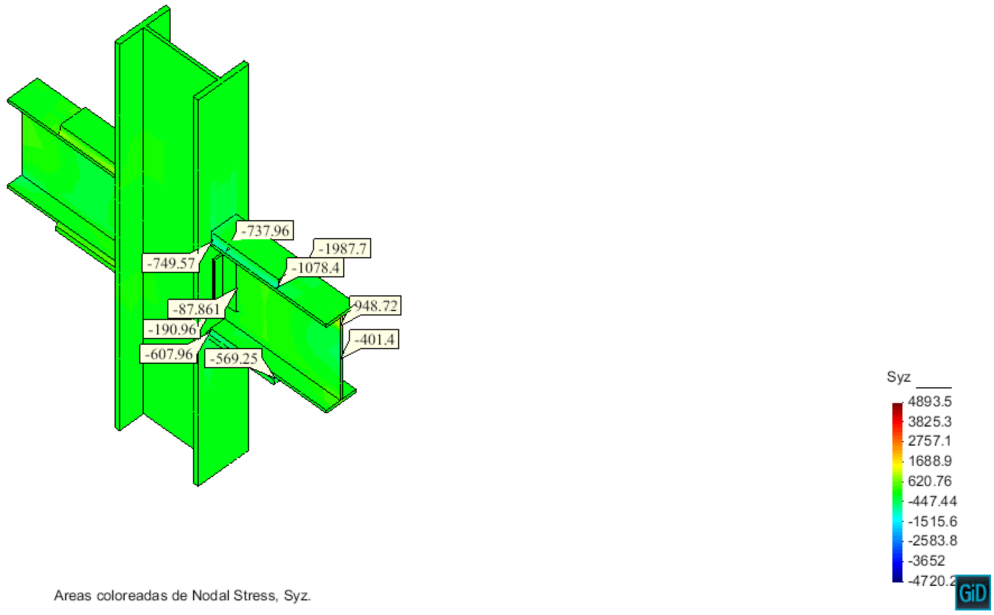


Figure 92: Shear stresses (XY plane, SXY) in M2-A36 N-QLF model.

Observations:

- Shear stresses along fillet welds does not exceed nominal strength.

Finite Element Analysis results: Model 2, Shear stresses S_{yz}.



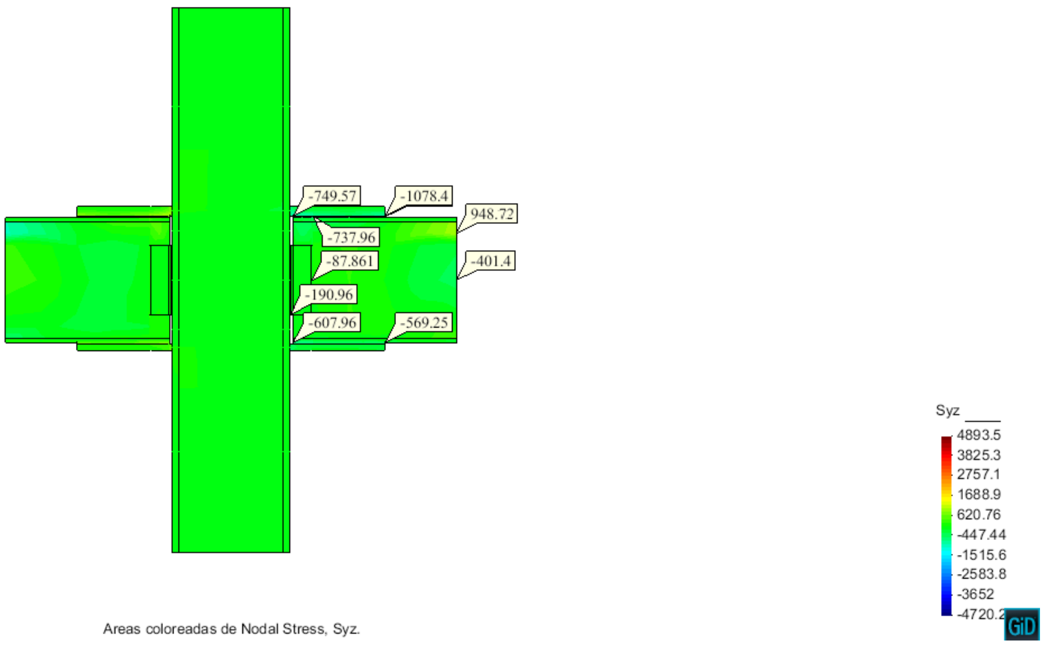
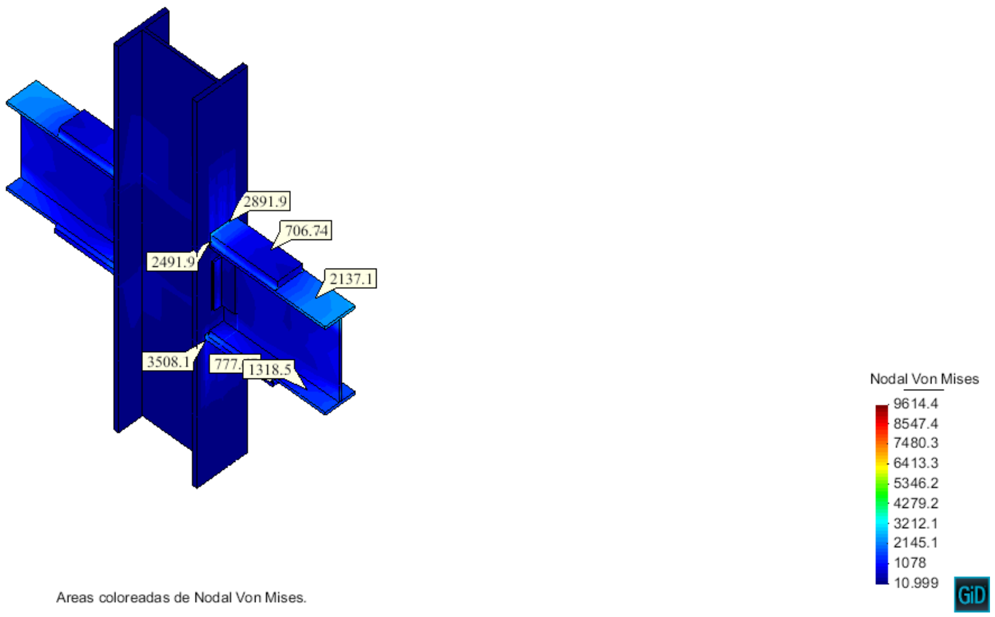


Figure 93: Shear stresses (YZ plane, SYZ) in M2-A36 N-QLF model.

Observations:

- Shear stresses along fillet welds around the gaps between upper plate and the beam arm exceed nominal strength about 25%, but resistance is even the theoretical.

Finite Element Analysis results: Model 2, Von Mises stresses VM_s.



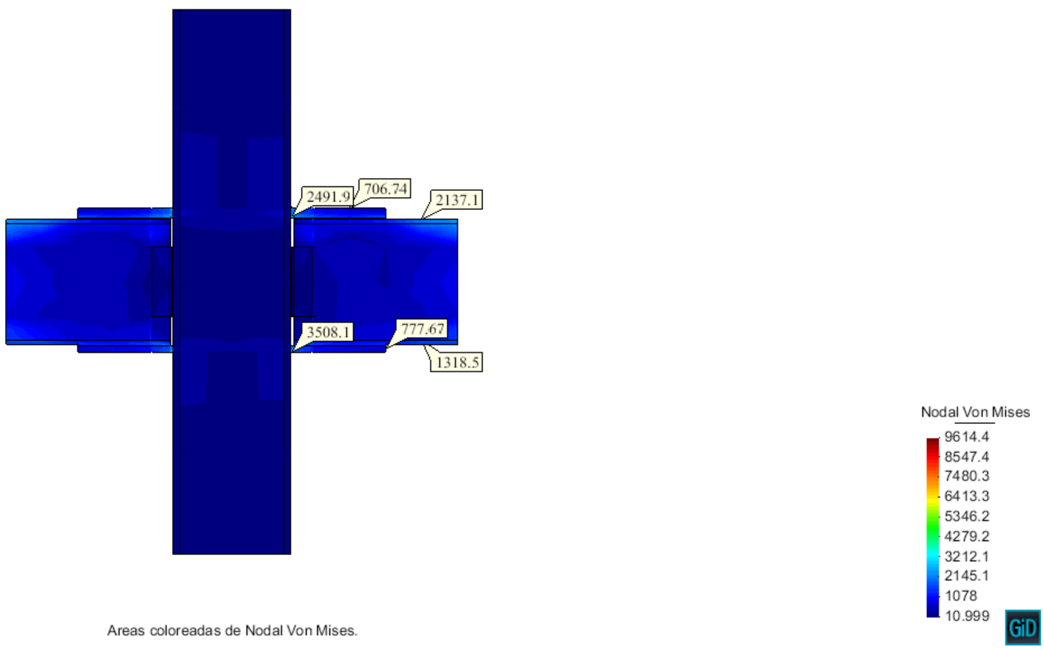


Figure 94: Von Mises stresses in M2-A36 N-QLF model.

Observations:

- The overstress in the lower plates is still yet under rupture tensile value.
- There is a perpendicular plane to the bending plates connected to column flanges overstressed.

6.2.4 Model 3.

Input Data

Name of model: M3-BM3-CL3-A36, (arbitrary model)
Beam: (BM_3) W24x55 — Column: (CL_3) W16x100 — Steel Designation: ASTM-A572 Gr. 50
F_y= $3515 kg/cm^2$ $(50ksi)$ — F_u= $4570 kg/cm^2$ $(65ksi)$ — Thickness governant: Column
The flange of beam fits in the flange of column $(0.85 b_{fc} > b_{fb})$.
From beam:

- M_{pf}: $4.76\ e+06\ kg \cdot cm$ flanges plastic moment contribution.
- M_{pw}: $2.90\ e+06\ kg \cdot cm$ web plastic moment contribution.
- M_p: $7.66\ e+06\ kg \cdot cm$ total plastic moment.
- M_y: $6.57\ e+06\ kg \cdot cm$ yield moment.
- $\frac{M_{pf}}{M_p} = 62.12\%$ percentage contribution of flanges respect to total plastic moment.
- $\frac{M_{pw}}{M_p} = 37.88\%$ percentage contribution of web respect to total plastic moment.

Design forces:

- $V_u = 3.77\ e+04\ kg$ ultimate shear load.
- $V_s = 2.69\ e+04\ kg$ service shear load.
- $M_u = 5.25\ e+06\ kg \cdot cm$ ultimate moment.
- $M_s = 3.75\ e+06\ kg \cdot cm$ service moment.

Rigid Upper Plate

From beam:

Flexural strength.		
$F(kg \cdot cm)$	LRFD (ϕM_n)	ASD $(\frac{M_n}{\Omega})$
Nominal moment	6.90e+06	4.59e+06

$F_{ut} = F_{uc} = 0.96\ e+05\ kg$ ultimate tension/compression force to design bending plates (according to M_u).
$F_{st} = F_{sc} = 0.63\ e+05\ kg$ service tension/compression force to design bending plates (according to M_s).

Resultant dimensions:

- $b_p = 15.56 cm$ $(6.125 in)$ width of plate.
- $t_p = 3.18 cm$ $(1.25 in)$ thickness of plate.
- $L_p = 35.00 cm$ $(14.00 in)$ length of plate.
- $\psi = 1.18$ shape factor.

Upper plate nominal force.		
$F(kg)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal yield force	1.12e+05	0.75e+05
Nominal rupture force	1.51e+05	1.00e+05

Fillet Welds for Rigid Plates

Results:

- $EXX/FX = 4900kg/cm^2$ $(70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.80cm$ minimum leg size of weld.
- $tw_{max} = 2.98cm$ maximum leg size of weld.
- $w_p = 1.00cm$ leg size of weld for design of project.
- $Lw_{req} = 61.50cm$ length of weld required.
- $Lw_p = 68.00cm$ length of weld to be adjusted to the project.
- $Lw_{eff} = 68.00cm$ effective length of proportioned weld.
- $R_{nw} = 2078.58kg/cm$ theoretical strength of weld.

Nominal strength of fillet weld.		
$F(kg/cm)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal strength	1558.94	1039.29

Nominal force due to fillet welds.		
$F(kg)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal force	1.06e+05	0.71e+05

Rigid Lower Plate

Results:

- $b_p = 21.59cm$ $(8.50in)$ width of plate.
- $t_p = 2.22cm$ $(0.875in)$ thickness of plate.
- $L_p = 35.00cm$ $(14.00in)$ length of plate.
- $r = 0.642cm$ gyration radius of plate.
- $k = 0.65$ effective length factor.
- $kL_p = 22.75cm$ effective length.
- $\frac{kL_p}{r} = 35.46$ slenderness ratio (adimensional).
- $F_e = 16.01\ e+03\ kg/cm^2$ Euler stress.
- $F_{cr} = 2.37\ e+03\ kg/cm^2$ critical stress.
- $P_n = 1.14\ e+05\ kg$ theoretical compression force resisted.

Compression force resisted by lower plate.		
$F(kg)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal force	1.02e+05	0.68e+05

Shear Design

Beam checking:

- $\frac{h}{t_w}$ = 59.75 slenderness ratio of web.
- k_v = 5.34 (a=0, no transverse stiffeners) adimensional value.
- C_{v1} = 1.00 adimensional value.
- $A_w = 60.14cm^2$ area of the web.

Shear strength of beam section.		
$F(kg)$	LRFD (ϕV_n)	ASD ($\frac{V_n}{\Omega}$)
Nominal shear resistance	11.42e+04	7.60e+04

Angles:

- Angle: $L3\ x\ 2\frac{1}{2}\ x\ \frac{3}{8}$
- $b_p = 6.35cm\ (2.50in)$ width of plate.
- $t_p = 0.95cm\ (0.375in)$ thickness of plate.
- $L_p\ or\ h = 38.00cm\ (15.00in)$ length of plate.

Strength of angles (the pair).		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal yield strength	9.92e+04	6.60e+04
Nominal rupture strength	13.32e+04	8.88e+04

Results for fillet welds of "back-to-back" angles:

- $EXX/FX = 4900kg/cm^2\ (70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.50cm$ minimum leg size of weld.
- $tw_{max} = 0.75cm$ maximum leg size of weld.
- $w_p = 0.50cm$ leg size of weld for design of project.
- $Lw_p = 71.12cm$ length of weld proportioned.
- $Lw_{eff} = 71.12cm$ effective length of proportioned weld.

Nominal strength of fillet welds for angles.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal strength	4.43e+04	2.96e+04

Additional quantities.		
Weight	*Length of fillet weld (total)*	*Length of groove weld (total)*
33.78 Kg	142.24 cm of 5.0 mm, and 136.00 cm of 10 mm	33.34 cm

Flanges and Webs with concentrated forces (Chapter J.10)

Flange local bending.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	3.25e+04	2.16e+04
Column	12.37e+04	8.23e+04

From column:

- $t_w = 1.49cm$ thickness of web.
- $k = 3.53cm$ eccentricity of W shape.
- $l_b = 4.22cm$ bearing length (min(t_p,t_{fb})+2w); where:
  - w is the leg size of weld project (w_p).
  - t_p is the thicker thickness of the bending plates.
  - t_{fb} is the thickness of the flange beam.

Web local yielding.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	11.05e+04	7.37e+04

From column:

- $t_f = 2.50cm$ thickness of flange.
- $h = 43.18cm$ depth of W shape.
- $l_b = 4.22cm$ bearing length.
- $Q_f = 1.00$ adimensional factor.

Web local crippling.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	16.51e+04	11.00e+04

Transverse stiffeners (Chapter G2, Section 3 from ANSI/AISC-360-16):

Feature	**Beam**	**Column**
h	59.94cm	43.18cm
t_w	1.00cm	1.49cm
$\frac{h}{t_w}$	59.75	29.06
$2.46\sqrt{\frac{E}{F_{yw}}}$	59.26	59.26
Requirement?	YES	NO

- $F_{yw} = 3515kg/cm^2$ yield stress.
- Transverse stiffeners **not necessary if**:

$$\frac{h}{t_w} < 2.46\sqrt{\frac{E}{F_{yw}}}$$

Stiffening/Continuity Plates

- $C_u = T_u$: 0.00
- A_{req}: $8.26cm^2$
- $\dfrac{b_{fb}}{3}$: $5.94cm$
- $0.56\sqrt{\dfrac{E}{F_y}}$: 15.90

For upper plate ($t_{up} = 3.18cm$ (1.25in)):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	33.53	34.37	10.56	7.63e+04	5.08e+04
Mid plate	16.76	34.37	5.28	3.81e+04	2.54e+04
Optimal	9.53	34.37	3.00	2.17e+04	1.44e+04

For lower plate ($t_{lp} = 2.22cm$ (0.875in)):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	23.47	34.37	10.56	5.34e+04	3.56e+04
Mid plate	11.73	34.37	5.28	2.67e+04	1.78e+04
Optimal	8.89	34.37	4.00	2.02e+04	1.35e+04

Note: strengths (kg) are expressed for just one plate.
For welds (two alternatives, it considers upper and lower edges):

- t_{w1} (weld leg size for alternative 1) = $0.30cm$ $(\frac{1}{8})''$
- t_{w2} (weld leg size for alternative 2) = $0.50cm$

Plate	$LRFD(\phi R_{n1})$	$ASD(\frac{R_{n1}}{\Omega})$	$LRFD(\phi R_{n2})$	$ASD(\frac{R_{n2}}{\Omega})$
FP/MP/OP	3.21e+04	2.14e+04	5.36e+04	3.57e+04

Observations

- Plates have a work performance up to 78% for yield (M_u), and about of 85% for ultimate design forces.
- Strength is delimited by yield of lower plate. Its resistance is very close to the fillet welds.
- A transverse stiffener is required (seated on beam). The area required for continuity plates can be adjusted with a minor thickness (it has to accomplish the allowable width), the chosen one was taken according to geometric conditions programmed for the spreadsheet and problem type code.

Finite Element Analysis results: Model 3, Displacements.

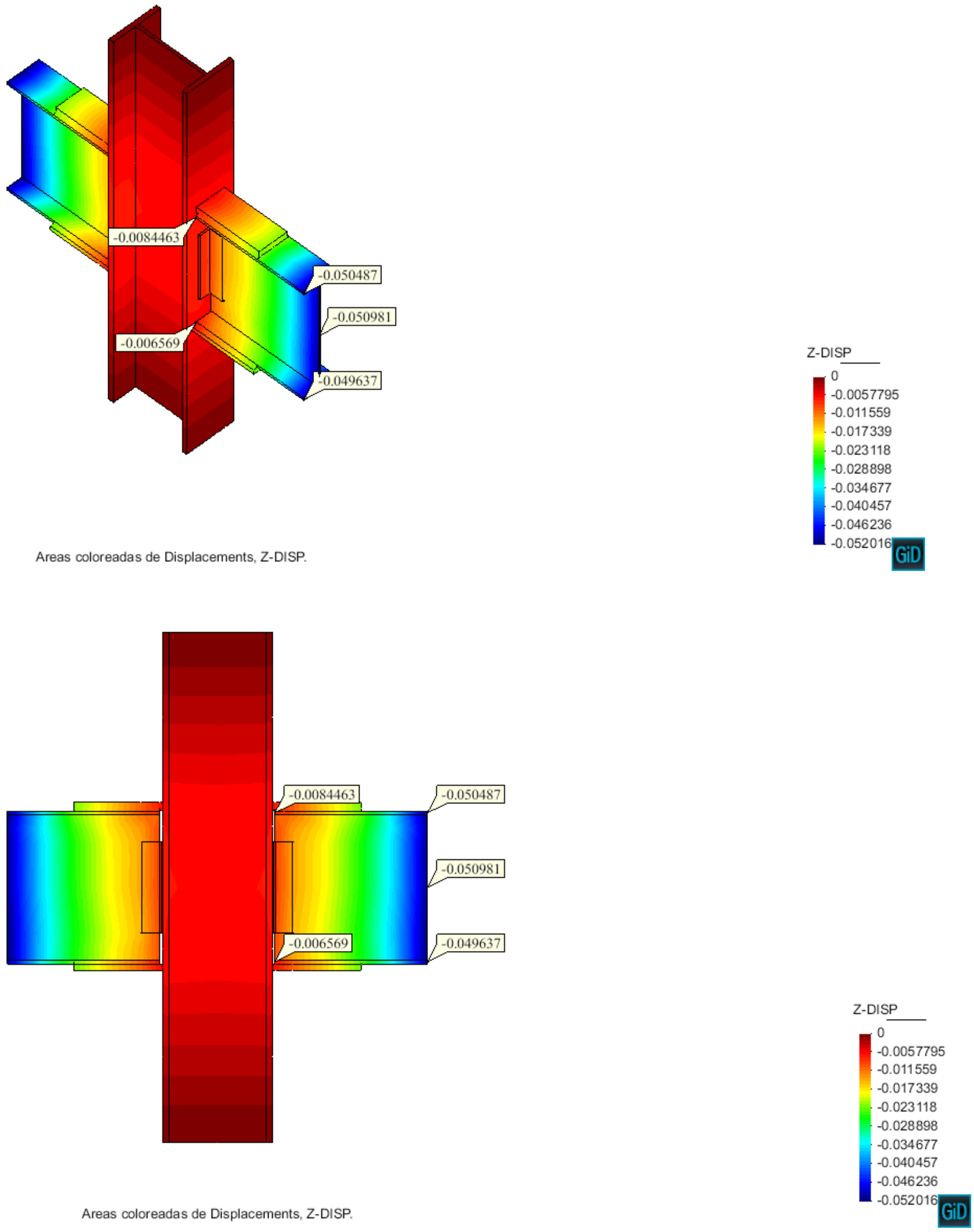


Figure 95: Vertical displacements (Z-axis) in M3-A36 N-QLF model.

Finite Element Analysis results: Model 3, Tension stresses S_{yy}.

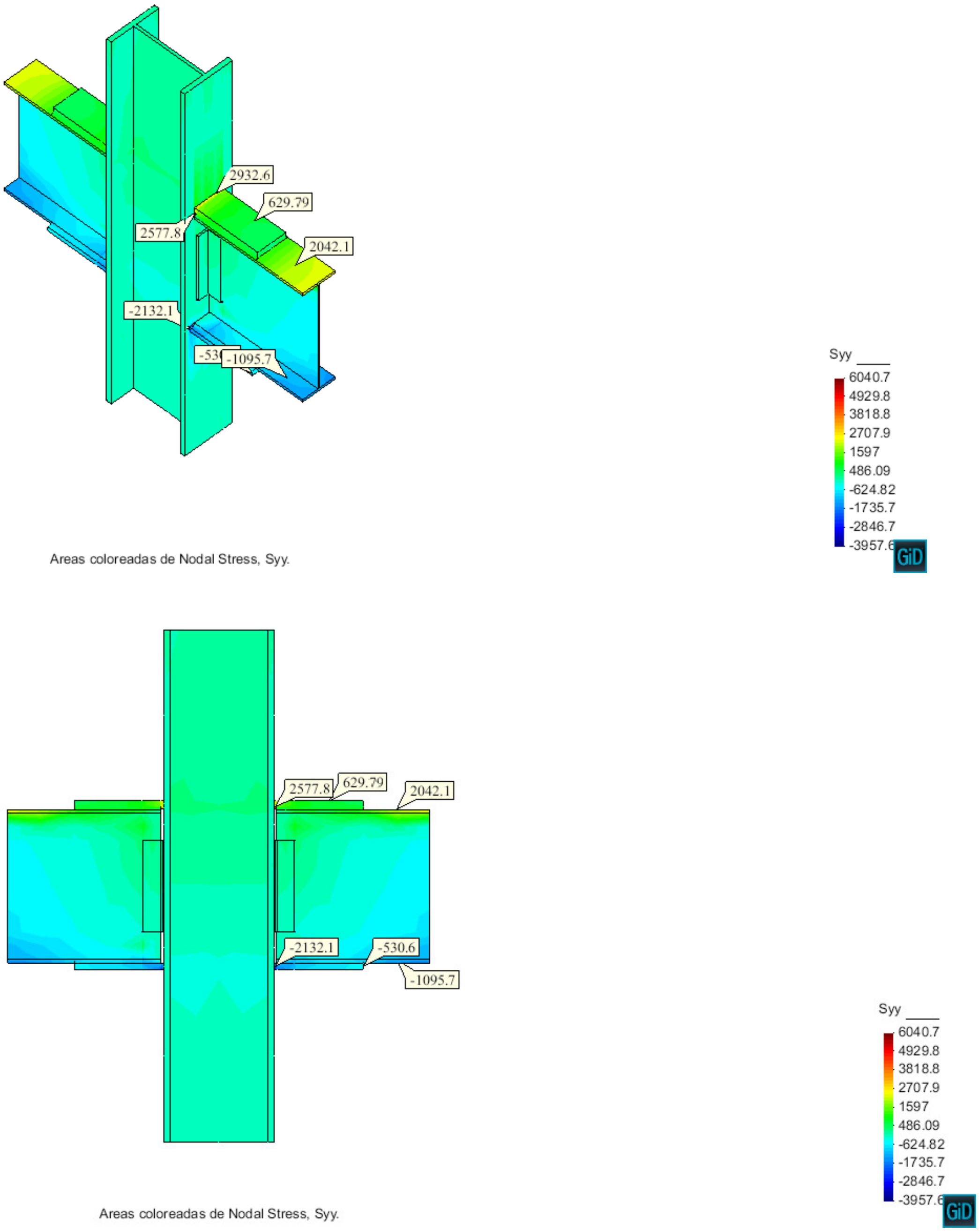
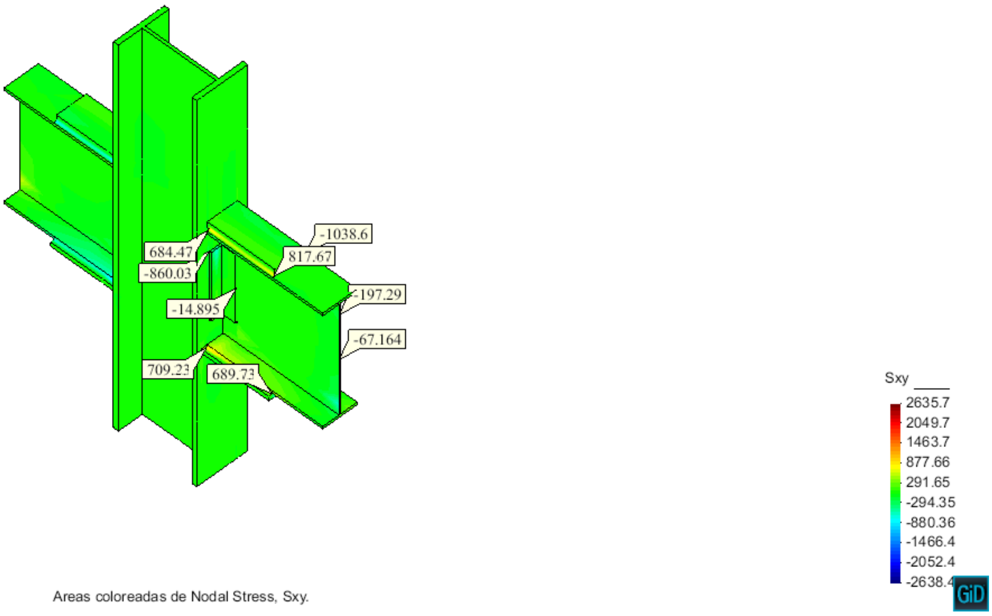


Figure 96: Axial stresses (Y-axis, SYY) in M3-A36 N-QLF model.

Observations:

- The upper fibers for the superior bending plates reach yield for an increase in stress of approximately of 16%. The extreme fibers found in opposite side to these faces are barely exceeding yield stress.
- Compression plates work under yield.

Finite Element Analysis results: Model 3, Shear stresses S_{xy}.



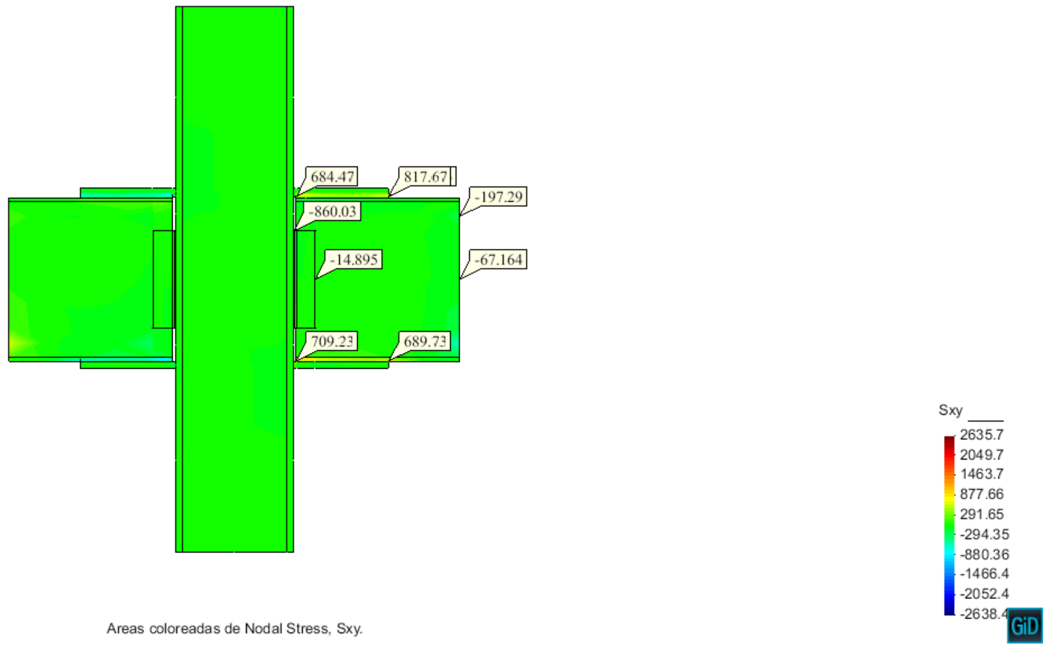


Figure 97: Shear stresses (XY plane, SXY) in M3-A36 N-QLF model.

Observations:

- All values are inside allowable parameters.

Finite Element Analysis results: Model 3, Shear stresses S_{yz}.

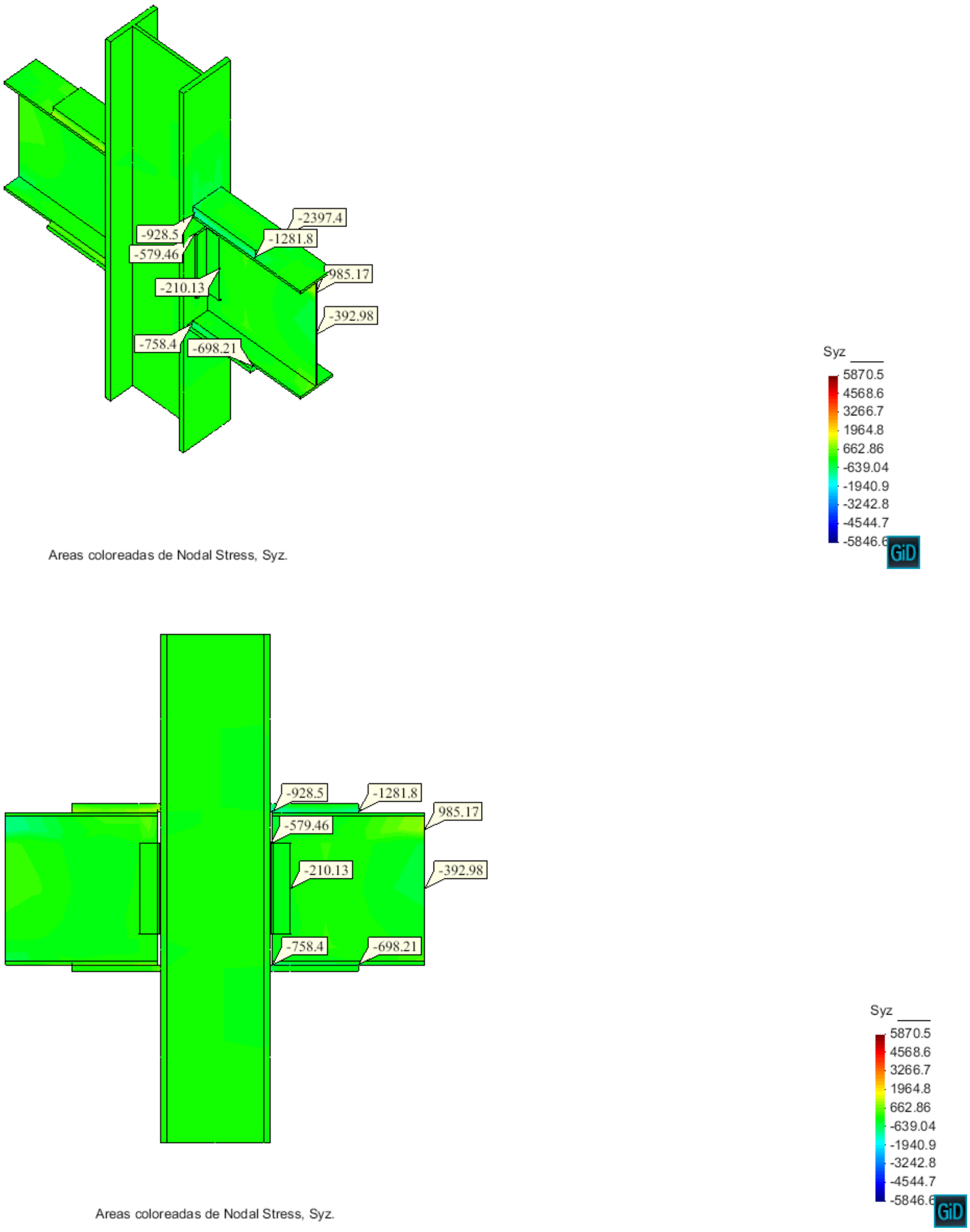


Figure 98: Shear stresses (YZ plane, SYZ) in M3-A36 N-QLF model.

Observations:

- All values are inside allowable parameters, just the inner surfaces in fillet welds between upper plate and beam (gap of 3mm) is exceeding the shear capacity.

Finite Element Analysis results: Model 3, Von Mises stresses VM_s.

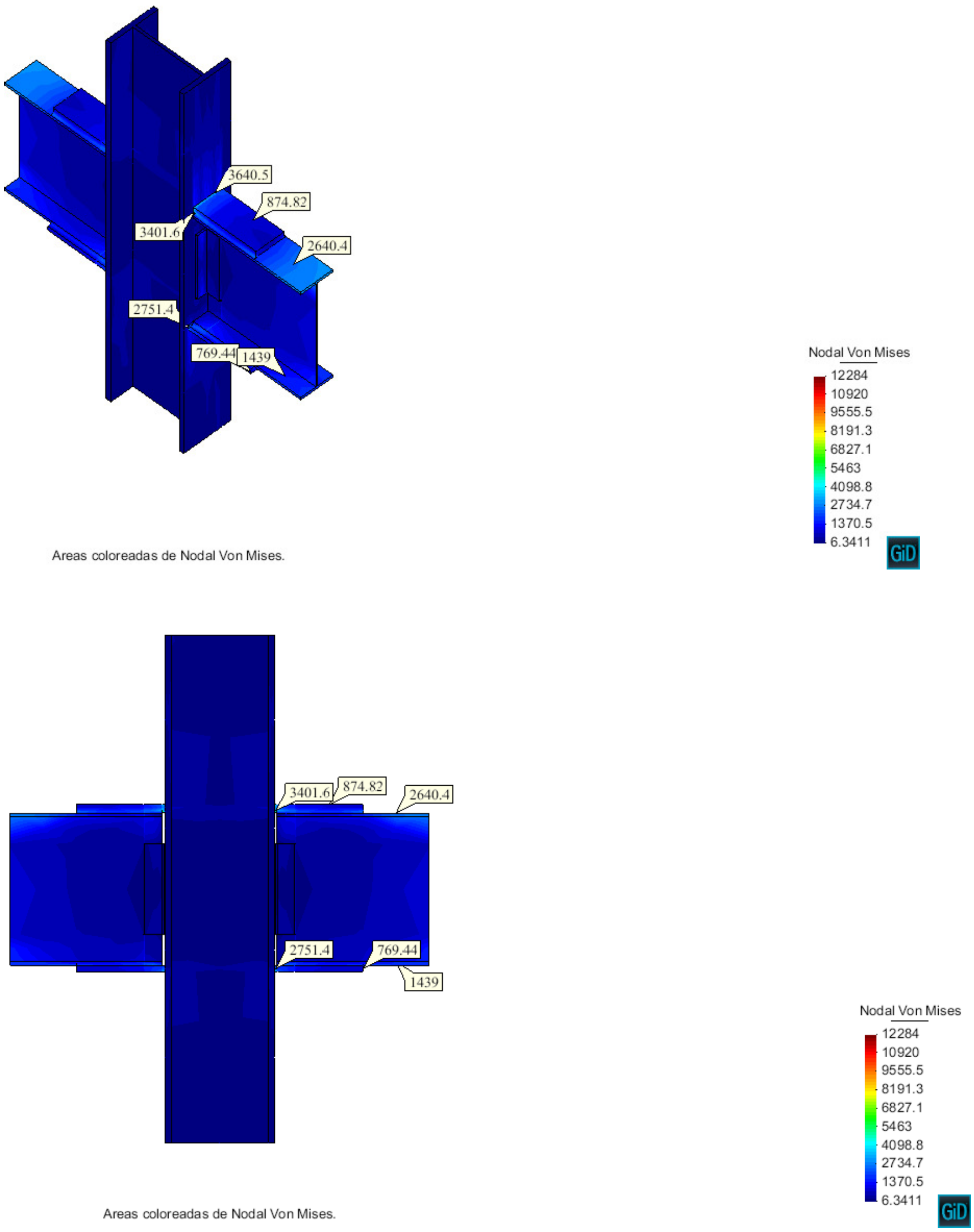


Figure 99: Von Mises stresses in M3-A36 N-QLF model.

7 Results: Union Plates ASTM A572 Gr. 50

7.1 Qualified Bending Moment Configurations

7.1.1 Model 0.

Input Data

Name of model: M0-BM0-CL0-A572GR50, (arbitrary model)
Beam: (BM_0) W12x30 — Column: (CL_0) W16x100 — Steel Designation: ASTM-A572 Gr. 50
F_y= $3515 kg/cm^2$ $(50 ksi)$ — F_u= $4570 kg/cm^2$ $(65 ksi)$ — Thickness governant: Column
The flange of beam fits in the flange of column $(0.85 b_{fc} > b_{fb})$.
From beam:

- M_{pf}: $1.99\ e+06\ kg \cdot cm$ flanges plastic moment contribution.
- M_{pw}: $0.49\ e+06\ kg \cdot cm$ web plastic moment contribution.
- M_p: $2.48\ e+06\ kg \cdot cm$ total plastic moment.
- M_y: $2.22\ e+06\ kg \cdot cm$ yield moment.
- $\frac{M_{pf}}{M_p} = 80.35\%$ percentage contribution of flanges respect to total plastic moment.
- $\frac{M_{pw}}{M_p} = 19.65\%$ percentage contribution of web respect to total plastic moment.

Design forces:

- $V_u = 1.29\ e+04\ kg$ ultimate shear load.
- $V_s = 0.92\ e+04\ kg$ service shear load.
- $M_u = 1.78\ e+06\ kg \cdot cm$ ultimate moment.
- $M_s = 1.27\ e+06\ kg \cdot cm$ service moment.

Rigid Upper Plate

From beam:

Flexural strength.		
$F(kg \cdot cm)$	LRFD (ϕM_n)	ASD $(\frac{M_n}{\Omega})$
Nominal moment	2.23e+06	1.49e+06

$F_{ut} = F_{uc} = 0.80\ e+05\ kg$ ultimate tension/compression force to design bending plates (according to M_u).
$F_{st} = F_{sc} = 0.41\ e+05\ kg$ service tension/compression force to design bending plates (according to M_s).

Resultant dimensions:

- $b_p = 14.29 cm$ $(5.625 in)$ width of plate.
- $t_p = 2.22 cm$ $(0.875 in)$ thickness of plate.
- $L_p = 30.00 cm$ $(12.00 in)$ length of plate.
- $\psi = 1.12$ shape factor.

Upper plate nominal force.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal yield force	1.00e+05	0.67e+05
Nominal rupture force	1.09e+05	0.73e+05

Fillet Welds for Rigid Plates

Results:

- $EXX/FX = 4900kg/cm^2$ $(70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.80cm$ minimum leg size of weld.
- $tw_{max} = 2.02cm$ maximum leg size of weld.
- $w_p = 1.00cm$ leg size of weld for design of project.
- $Lw_{req} = 51.00cm$ length of weld required.
- $Lw_p = 58.00cm$ length of weld to be adjusted to the project.
- $Lw_{eff} = 58.00cm$ effective length of proportioned weld.
- $R_{nw} = 2078.58kg/cm$ theoretical strength of weld.

Nominal strength of fillet weld.		
$F(kg/cm)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal strength	1558.94	1039.29

Nominal force due to fillet welds.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal force	0.90e+05	0.60e+05

Rigid Lower Plate

Results:

- $b_p = 21.59cm$ $(8.50in)$ width of plate.
- $t_p = 1.91cm$ $(0.75in)$ thickness of plate.
- $L_p = 30.00cm$ $(12.00in)$ length of plate.
- $r = 0.55cm$ gyration radius of plate.
- $k = 0.65$ effective length factor.
- $kL_p = 19.50cm$ effective length.
- $\frac{kL_p}{r} = 35.46$ slenderness ratio (adimensional).
- $F_e = 16.01\ e+03\ kg/cm^2$ Euler stress.
- $F_{cr} = 3.21\ e+03\ kg/cm^2$ critical stress.
- $P_n = 1.32\ e+05\ kg$ theoretical compression force resisted.

Compression force resisted by lower plate.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal force	1.19e+05	0.79e+05

Shear Design

Beam checking:

- $\frac{h}{t_w}$ = 47.31 slenderness ratio of web.
- k_v = 5.34 (a=0, no transverse stiffeners) adimensional value.
- C_{v1} = 1.00 adimensional value.
- $A_w = 20.63cm^2$ area of the web.

Shear strength of beam section.		
$F(kg)$	LRFD (ϕV_n)	ASD ($\frac{V_n}{\Omega}$)
Nominal shear resistance	3.92e+04	2.60e+04

Shear plates:

- $b_p = 10.16cm$ $(4.00in)$ width of plate.
- $t_p = 0.95cm$ $(0.375in)$ thickness of plate.
- L_p or $h = 15.24cm$ $(6.00in)$ length of plate.

Strength of shear plates (the pair).		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal yield strength	5.51e+04	3.67e+04
Nominal rupture strength	5.97e+04	3.98e+04

Results for fillet welds of shear plates:

- $EXX/FX = 4900kg/cm^2$ $(70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.50cm$ minimum leg size of weld.
- $tw_{max} = 0.75cm$ maximum leg size of weld.
- $w_p = 0.50cm$ leg size of weld for design of project.
- $Lw_p = 30.48cm$ length of weld proportioned.
- $Lw_{eff} = 30.48cm$ effective length of proportioned weld.

Nominal strength of fillet welds for shear plates.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal strength	2.38e+04	1.58e+04

Additional quantities.		
Weight	*Length of fillet weld (total)*	*Length of groove weld (total)*
19.49 Kg	30.48 cm of 5 mm, and 116.00 cm of 10 mm	35.88 cm

Flanges and Webs with concentrated forces (Chapter J.10)

Flange local bending.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	2.47e+04	1.64e+04
Column	12.37e+04	8.23e+04

From column:

- t_w = $1.49cm$ thickness of web.
- k = $3.53cm$ eccentricity of W shape.
- l_b = $3.91cm$ bearing length (min(t_p,t_{fb})+2w); where:
  - w is the leg size of weld project (w_p).
  - t_p is the thicker thickness of the bending plates.
  - t_{fb} is the thickness of the flange beam.

Web local yielding.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	10.74e+04	7.16e+04

From column:

- t_f = $2.50cm$ thickness of flange.
- h = $43.18cm$ depth of W shape.
- l_b = $3.91cm$ bearing length.
- Q_f = 1.00 adimensional factor.

Web local crippling.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	16.36e+04	10.91e+04

Transverse stiffeners (Chapter G2, Section 3 from ANSI/AISC-360-16):

Feature	**Beam**	**Column**
h	31.24cm	43.18cm
t_w	0.66cm	1.49cm
$\frac{h}{t_w}$	47.31	29.06
$2.46\sqrt{\frac{E}{F_{yw}}}$	59.26	59.26
Requirement?	NO	NO

- F_{yw} = $3515kg/cm^2$ yield stress.
- Transverse stiffeners **not necessary if**:

$$\frac{h}{t_w} < 2.46\sqrt{\frac{E}{F_{yw}}}$$

Stiffening/Continuity Plates

- $C_u = T_u$: 0.00
- A_{req}: 0.00
- $\frac{b_{fb}}{3}$: $5.52cm$
- $0.56\sqrt{\frac{E}{F_y}}$: 13.49

For upper plate ($t_{up} = 2.22cm$ $(0.875in)$):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	23.47	34.37	10.56	7.43e+04	4.94e+04
Mid plate	11.73	34.37	5.28	3.71e+04	2.47e+04
Optimal	0.00	34.37	0.00	0.00	0.00

For lower plate ($t_{lp} = 1.91cm$ $(0.75in)$):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	20.12	34.37	10.56	6.36e+04	4.23e+04
Mid plate	10.06	34.37	5.28	3.18e+04	2.11e+04
Optimal	0.00	34.37	0.00	0.00	0.00

Note: strengths (kg) are expressed for just one plate.
For welds (two alternatives, it considers upper and lower edges):

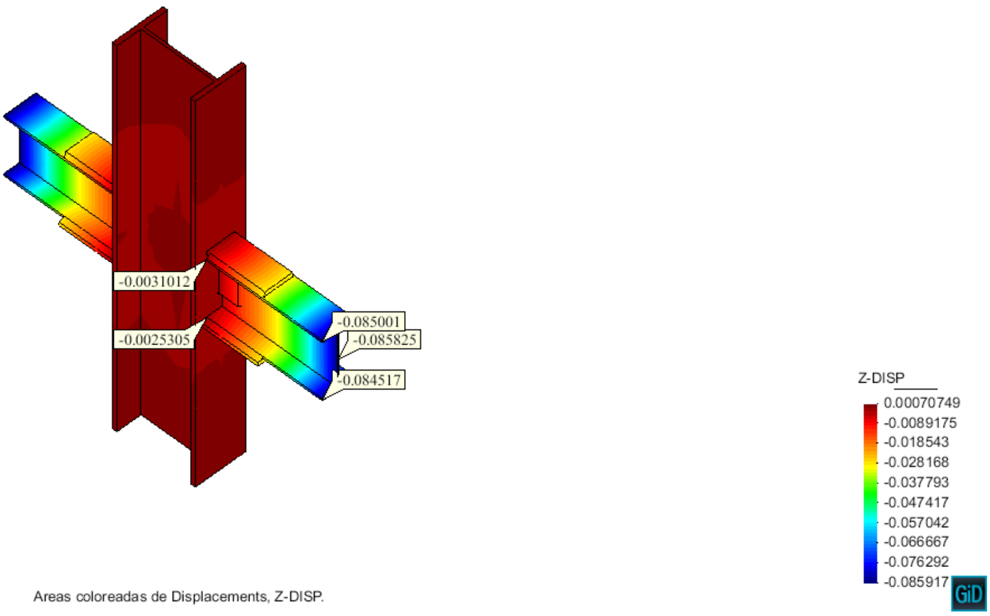
- t_{w1} (weld leg size for alternative 1) $= 0.30cm$ $(\frac{1}{8})''$
- t_{w2} (weld leg size for alternative 2) $= 0.50cm$

Plate	$LRFD(\phi R_{n1})$	$ASD(\frac{R_{n1}}{\Omega})$	$LRFD(\phi R_{n2})$	$ASD(\frac{R_{n2}}{\Omega})$
FP/MP/OP	3.21e+04	2.14e+04	5.36e+04	3.57e+04

Observations

- Strength is ruled by fillet welds capacity.
- The upper plate has an efficiency up to 70%, and the lower up to 60%.
- The fillet welds for shear plates can resist the force design almost twice magnitude.
- Stiffener elements are not needed.

Finite Element Analysis results: Model 0, Displacements.



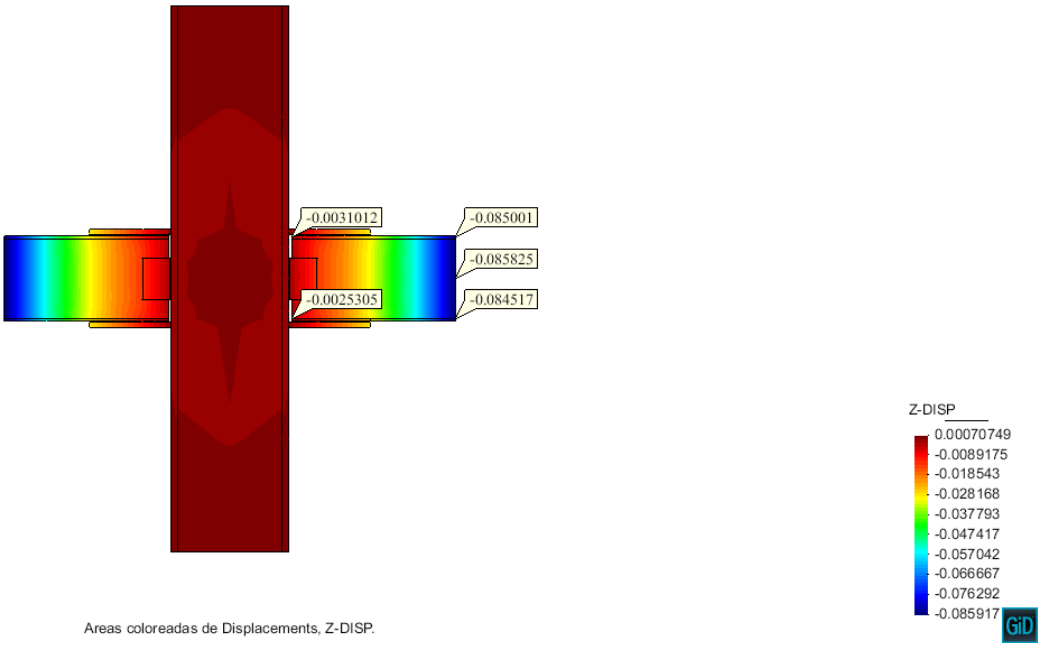


Figure 100: Vertical displacements (Z-axis) in M0-A572 conventional model.

Finite Element Analysis results: Model 0, Tension stresses S_{yy}.

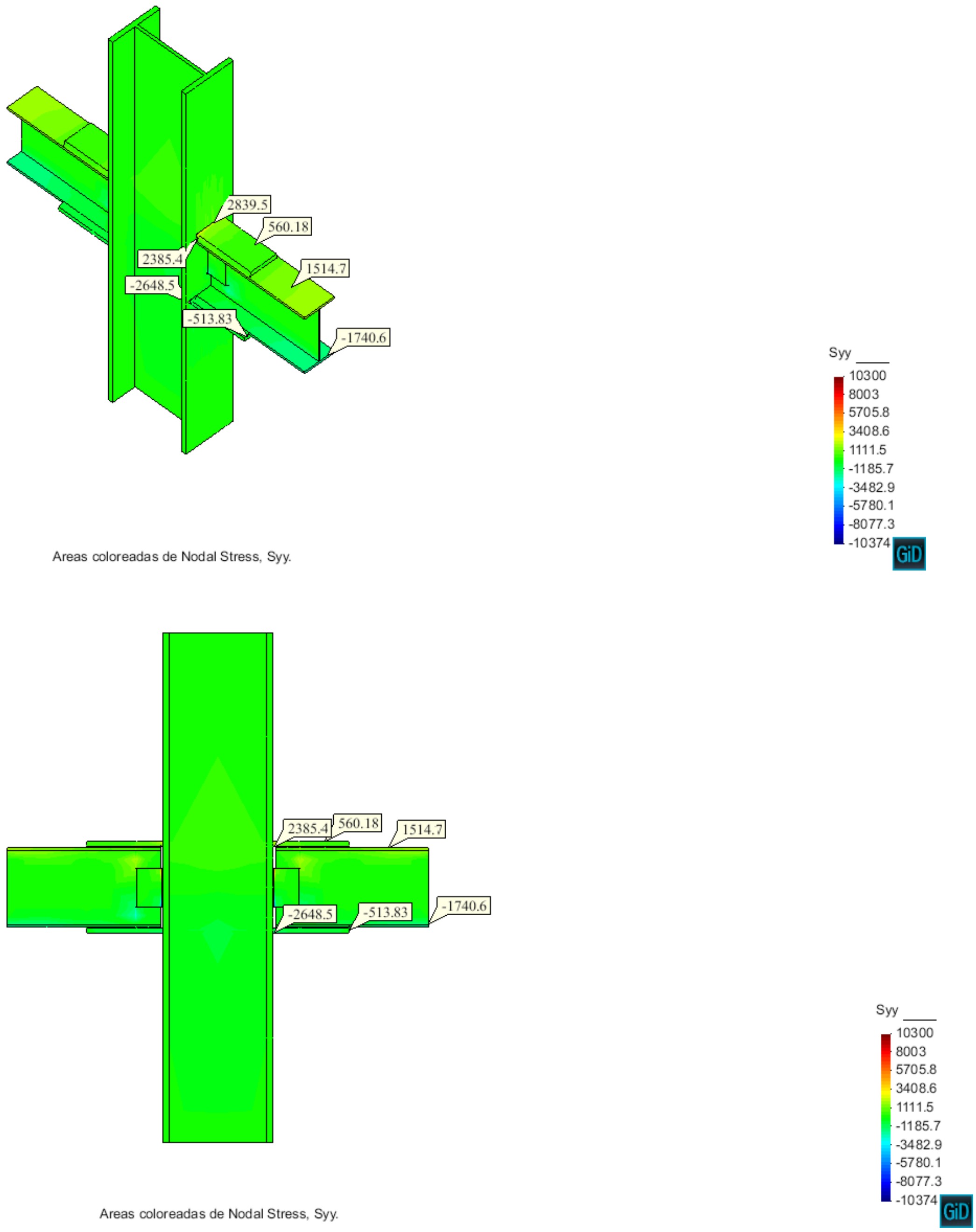


Figure 101: Axial stresses (Y-axis, SYY) in M0-A572 conventional model.

Observations:

- Upper and lower bending plates have a good performance to axial stress with values below yield.

Finite Element Analysis results: Model 0, Shear stresses S_{xy}.

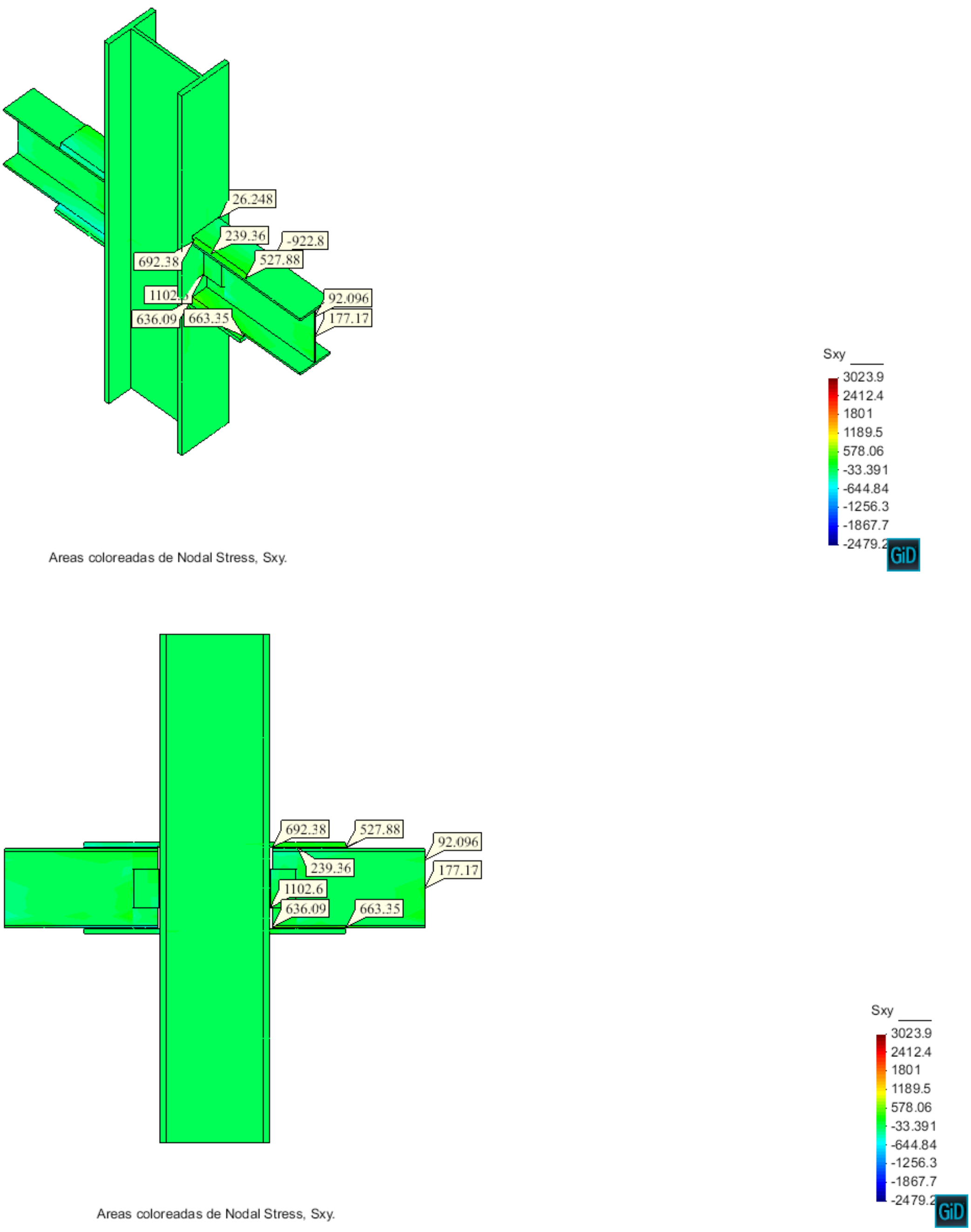
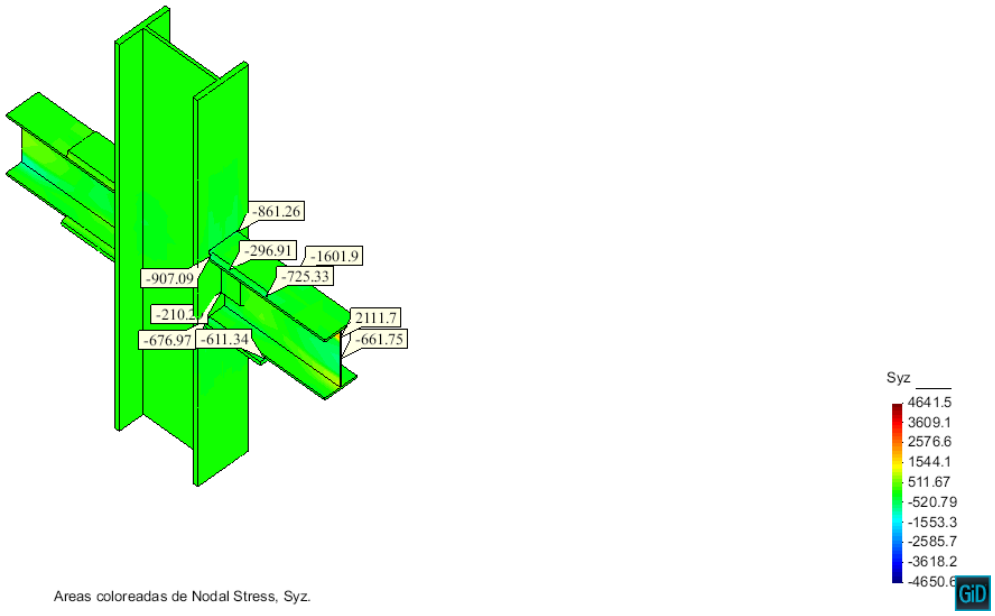


Figure 102: Shear stresses (XY plane, SXY) in M0-A572 conventional model.

Observations:

- All values are below an allowable limit.

Finite Element Analysis results: Model 0, Shear stresses S_{yz}.



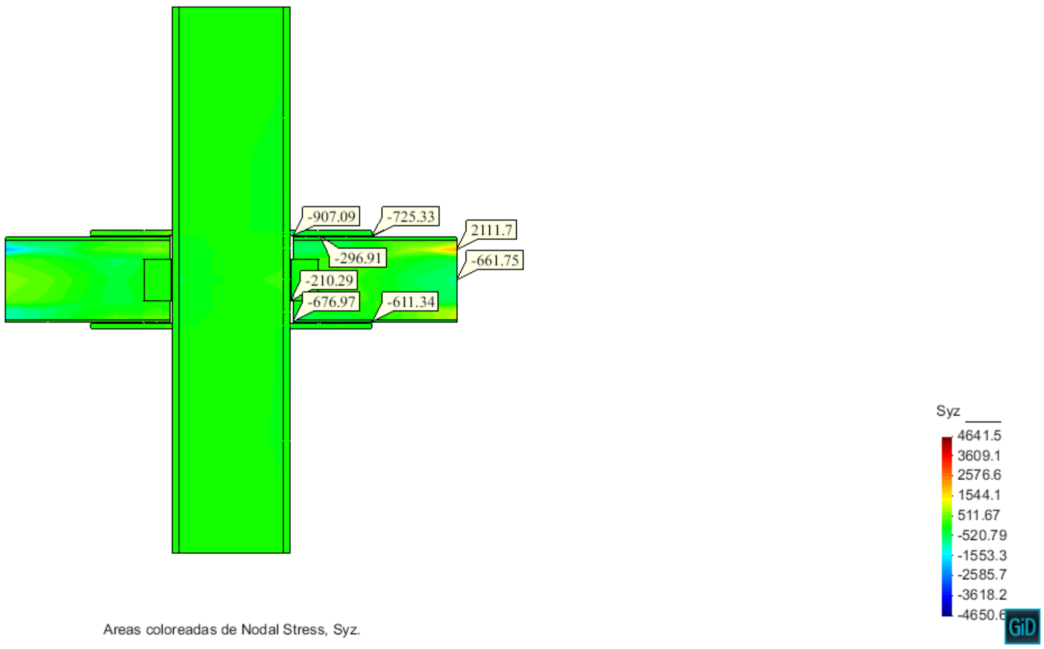
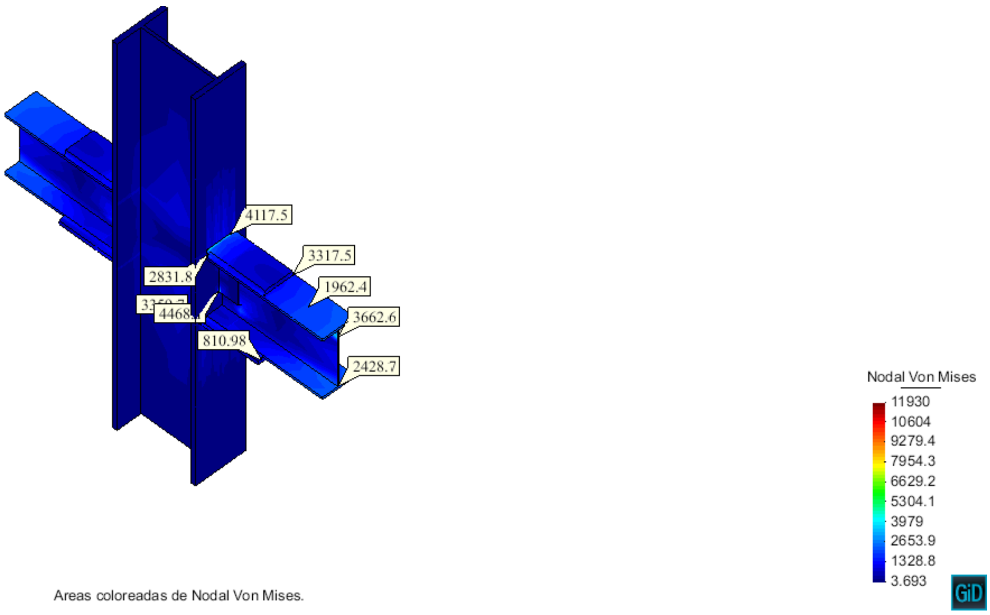


Figure 103: Shear stresses (YZ plane, SYZ) in M0-A572 conventional model.

Observations:

- The extremities of upper fillet welds in gaps barely exceed design shear stress for a minimal percentage (2.5%).
- The areas in the web near to the flanges of the beam have a considerable concentration of shear stress.

Finite Element Analysis results: Model 0, Von Mises stresses VM_s.



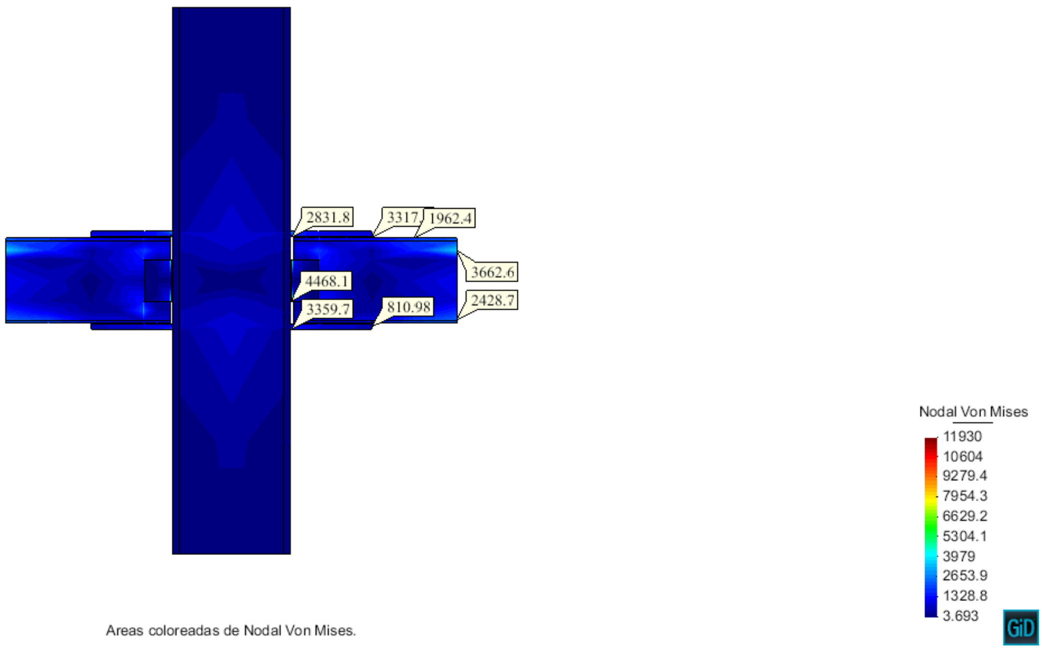


Figure 104: Von Mises stresses in M0-A572 conventional model.

7.1.2 Model 1.

Input Data

Name of model: M1-BM1-CL1-A572GR50, (model for offices building)
Beam: (BM_1) W16x45 — Column: (CL_1) W16x100 — Steel Designation: ASTM-A572 Gr. 50
F_y= $3515kg/cm^2$ $(50ksi)$ — F_u= $4570kg/cm^2$ $(65ksi)$ — Thickness governant: Column
The flange of beam fits in the flange of column $(0.85b_{fc} > b_{fb})$.
From beam:

- M_{pf}: $3.62\ e+06\ kg \cdot cm$ flanges plastic moment contribution.
- M_{pw}: $1.11\ e+06\ kg \cdot cm$ web plastic moment contribution.
- M_p: $4.74\ e+06\ kg \cdot cm$ total plastic moment.
- M_y: $4.19\ e+06\ kg \cdot cm$ yield moment.
- $\dfrac{M_{pf}}{M_p} = 76.50\%$ percentage contribution of flanges respect to total plastic moment.
- $\dfrac{M_{pw}}{M_p} = 23.50\%$ percentage contribution of web respect to total plastic moment.

Design forces:

- $V_u = 3.01\ e+04\ kg$ ultimate shear load.
- $V_s = 1.74\ e+04\ kg$ service shear load.
- $M_u = 3.81\ e+06\ kg \cdot cm$ ultimate moment.
- $M_s = 2.26\ e+06\ kg \cdot cm$ service moment.

Rigid Upper Plate

From beam:

Flexural strength.		
$F(kg \cdot cm)$	LRFD (ϕM_n)	ASD $(\frac{M_n}{\Omega})$
Nominal moment	4.26e+06	2.84e+06

$F_{ut} = F_{uc} = 1.16\ e+05\ kg$ ultimate tension/compression force to design bending plates (according to M_u).
$F_{st} = F_{sc} = 0.55\ e+05\ kg$ service tension/compression force to design bending plates (according to M_s).

Resultant dimensions:

- $b_p = 15.56cm\ (6.125in)$ width of plate.
- $t_p = 3.18cm\ (1.25in)$ thickness of plate.
- $L_p = 40.00cm\ (14.00in)$ length of plate.
- $\psi = 1.13$ shape factor.

Upper plate nominal force.		
$F(kg)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal yield force	1.56e+05	1.04e+05
Nominal rupture force	1.69e+05	1.12e+05

Fillet Welds for Rigid Plates

Results:

- $EXX/FX = 4900 kg/cm^2$ $(70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.80cm$ minimum leg size of weld.
- $tw_{max} = 2.98cm$ maximum leg size of weld.
- $w_p = 1.00cm$ leg size of weld for design of project.
- $Lw_{req} = 74.31cm$ length of weld required.
- $Lw_p = 78.00cm$ length of weld to be adjusted to the project.
- $Lw_{eff} = 78.00cm$ effective length of proportioned weld.
- $R_{nw} = 2078.58kg/cm$ theoretical strength of weld.

Nominal strength of fillet weld.		
$F(kg/cm)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal strength	1558.94	1039.29

Nominal force due to fillet welds.		
$F(kg)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal force	1.21e+05	0.81e+05

Rigid Lower Plate

Results:

- $b_p = 21.59cm$ $(8.50in)$ width of plate.
- $t_p = 2.22cm$ $(0.875in)$ thickness of plate.
- $L_p = 40.00cm$ $(16.00in)$ length of plate.
- $r = 0.642cm$ gyration radius of plate.
- $k = 0.65$ effective length factor.
- $kL_p = 26.00cm$ effective length.
- $\frac{kL_p}{r} = 40.52$ slenderness ratio (adimensional).
- $F_e = 12.26\ e+03\ kg/cm^2$ Euler stress.
- $F_{cr} = 3.11\ e+03\ kg/cm^2$ critical stress.
- $P_n = 1.50\ e+05\ kg$ theoretical compression force resisted.

Compression force resisted by lower plate.		
$F(kg)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal force	1.35e+05	0.90e+05

Shear Design

Beam checking:

- $\frac{h}{t_w} = 46.67$ slenderness ratio of web.
- $k_v = 5.34$ (a=0, no transverse stiffeners) adimensional value.
- $C_{v1} = 1.00$ adimensional value.
- $A_w = 35.84cm^2$ area of the web.

Shear strength of beam section.		
$F(kg)$	LRFD (ϕV_n)	ASD ($\frac{V_n}{\Omega}$)
Nominal shear resistance	6.80e+04	4.53e+04

Shear plates:

- $b_p = 10.16cm$ $(4.00in)$ width of plate.
- $t_p = 0.95cm$ $(0.375in)$ thickness of plate.
- L_p *or* $h = 22.86cm$ $(9.00in)$ length of plate.

Strength of shear plates (the pair).		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal yield strength	8.27e+04	5.50e+04
Nominal rupture strength	8.96e+04	5.97e+04

Results for fillet welds of shear plates:

- $EXX/FX = 4900kg/cm^2$ $(70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.50cm$ minimum leg size of weld.
- $tw_{max} = 0.75cm$ maximum leg size of weld.
- $w_p = 0.50cm$ leg size of weld for design of project.
- $Lw_p = 45.72cm$ length of weld proportioned.
- $Lw_{eff} = 45.72cm$ effective length of proportioned weld.

Nominal strength of fillet welds for shear plates.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal strength	3.56e+04	2.38e+04

Additional quantities.		
Weight	*Length of fillet weld (total)*	*Length of groove weld (total)*
34.05 Kg	45.72 cm of 5 mm, and 156.00 cm of 10 mm	37.15 cm

Flanges and Webs with concentrated forces (Chapter J.10)

Flange local bending.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	4.07e+04	2.71e+04
Column	12.37e+04	8.23e+04

From column:

- $t_w = 1.49cm$ thickness of web.
- $k = 3.53cm$ eccentricity of W shape.
- $l_b = 4.22cm$ bearing length (min(t_p,t_{fb})+2w); where:
  - w is the leg size of weld project (w_p).
  - t_p is the thicker thickness of the bending plates.
  - t_{fb} is the thickness of the flange beam.

Web local yielding.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	10.90e+04	7.27e+04

From column:

- $t_f = 2.50cm$ thickness of flange.
- $h = 43.18cm$ depth of W shape.
- $l_b = 4.22cm$ bearing length.
- $Q_f = 1.00$ adimensional factor.

Web local crippling.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	16.51e+04	11.00e+04

Transverse stiffeners (Chapter G2, Section 3 from ANSI/AISC-360-16):

Feature	**Beam**	**Column**
h	40.89cm	43.18cm
t_w	0.88cm	1.49cm
$\frac{h}{t_w}$	46.67	29.06
$2.46\sqrt{\frac{E}{F_{yw}}}$	59.26	59.26
Requirement?	NO	NO

- $F_{yw} = 3515kg/cm^2$ yield stress.
- Transverse stiffeners **not necessary if**:

$$\frac{h}{t_w} < 2.46\sqrt{\frac{E}{F_{yw}}}$$

Stiffening/Continuity Plates

- $C_u = T_u$: 0.00
- A_{req}: $2.15cm^2$
- $\frac{b_{fb}}{3}$: $5.96cm$
- $0.56\sqrt{\frac{E}{F_y}}$: 13.49

For upper plate ($t_{up} = 3.18cm$ $(1.25in)$):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	33.53	34.37	10.56	10.60e+04	7.06e+04
Mid plate	16.76	34.37	5.28	5.80e+04	3.53e+04
Optimal	0.00	34.37	0.00	0.00	0.00

For lower plate ($t_{lp} = 2.22cm$ $(0.875in)$):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	23.47	34.37	10.56	7.42e+04	4.94e+04
Mid plate	11.73	34.37	5.28	3.71e+04	2.47e+04
Optimal	0.00	34.37	0.00	0.00	0.00

Note: strengths (kg) are expressed for just one plate.
For welds (two alternatives, it considers upper and lower edges):

- t_{w1} (weld leg size for alternative 1) $= 0.30cm$ $(\frac{1}{8})''$
- t_{w2} (weld leg size for alternative 2) $= 0.50cm$

Plate	$LRFD(\phi R_{n1})$	$ASD(\frac{R_{n1}}{\Omega})$	$LRFD(\phi R_{n2})$	$ASD(\frac{R_{n2}}{\Omega})$
FP/MP/OP	3.21e+04	2.14e+04	5.36e+04	3.57e+04

Observations

- Both bending plates reach a performance of 75% compared to the total yield strength.
- The resistance is delimited by fillet welds.
- Shear welds have a 15% capacity extra.
- The ratio percentage of ultimate shear taken and nominal shear resisted is 44%.
- Additional stiffening plates are not demanded.

Finite Element Analysis results: Model 1, Displacements.

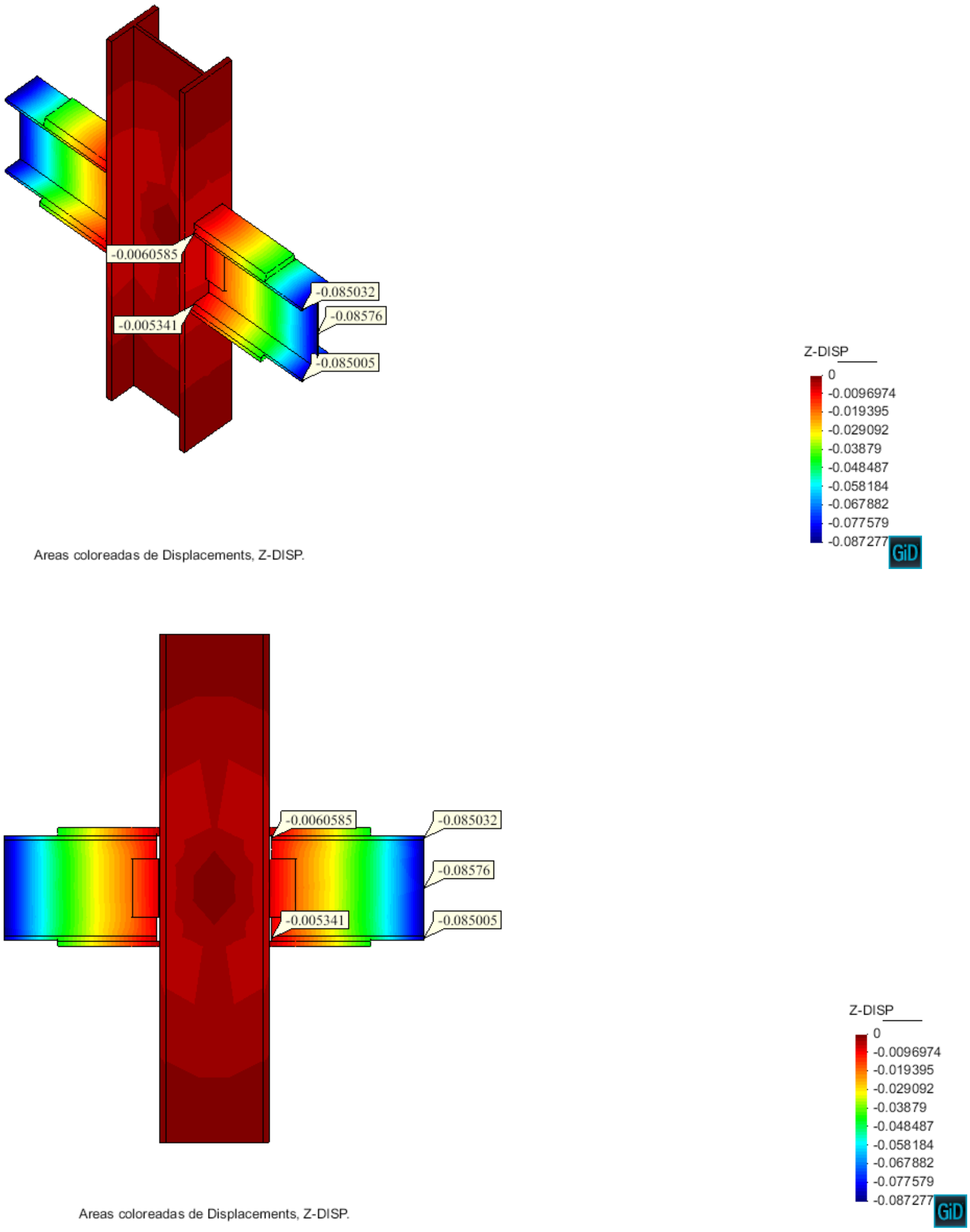
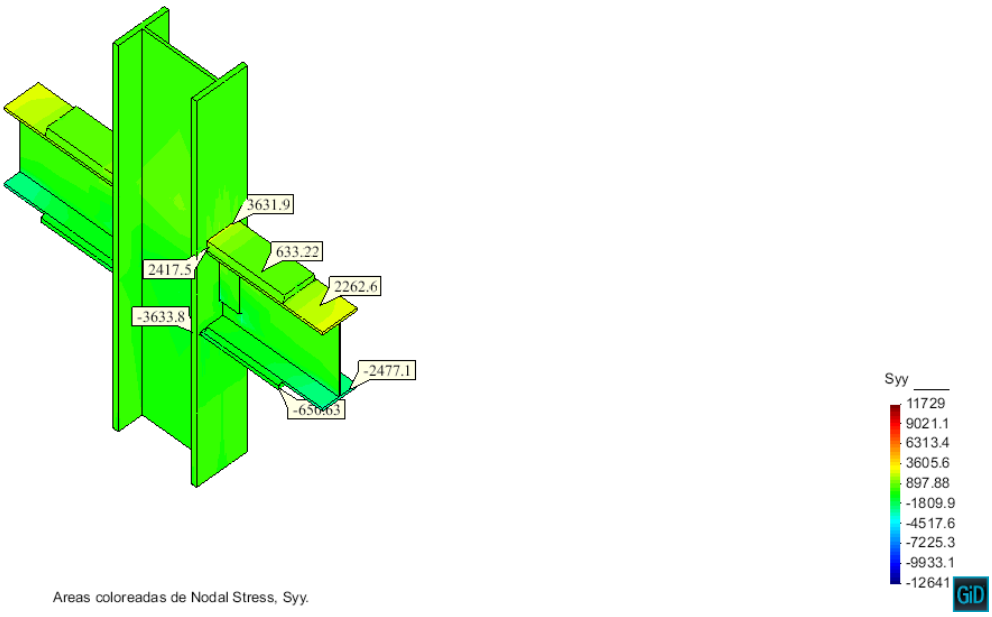


Figure 105: Vertical displacements (Z-axis) in M1-A572 conventional model.

Finite Element Analysis results: Model 1, Tension stresses S_{yy}.



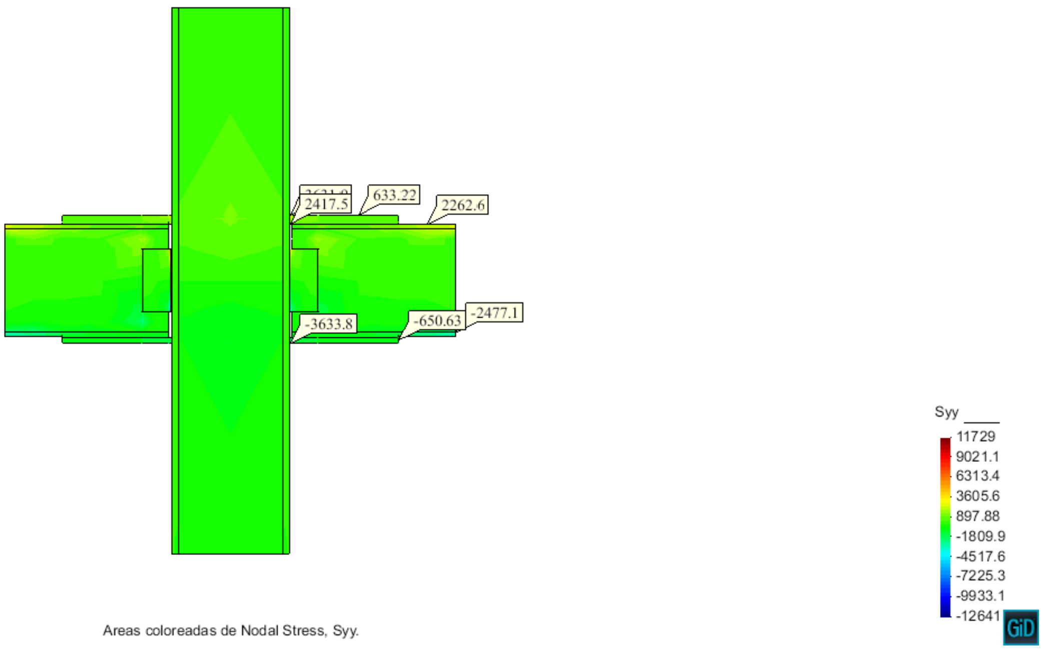


Figure 106: Axial stresses (Y-axis, SYY) in M1-A572 conventional model.

Observations:

- Bending plates are exceeding yield barely by a 3%.
- The faces of rigid plates between the beam located have values under σ_y.

Finite Element Analysis results: Model 1, Shear stresses S_{xy}.

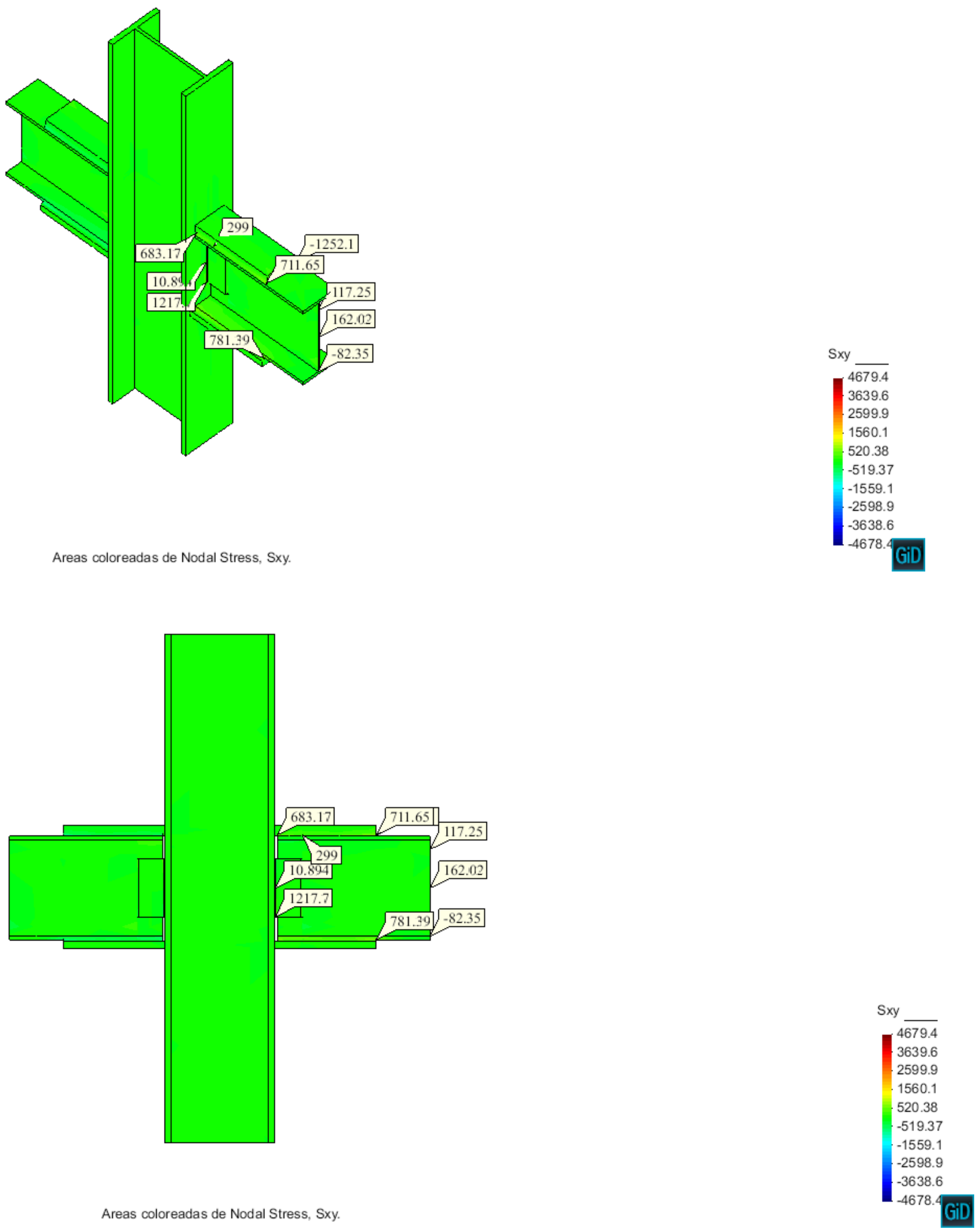
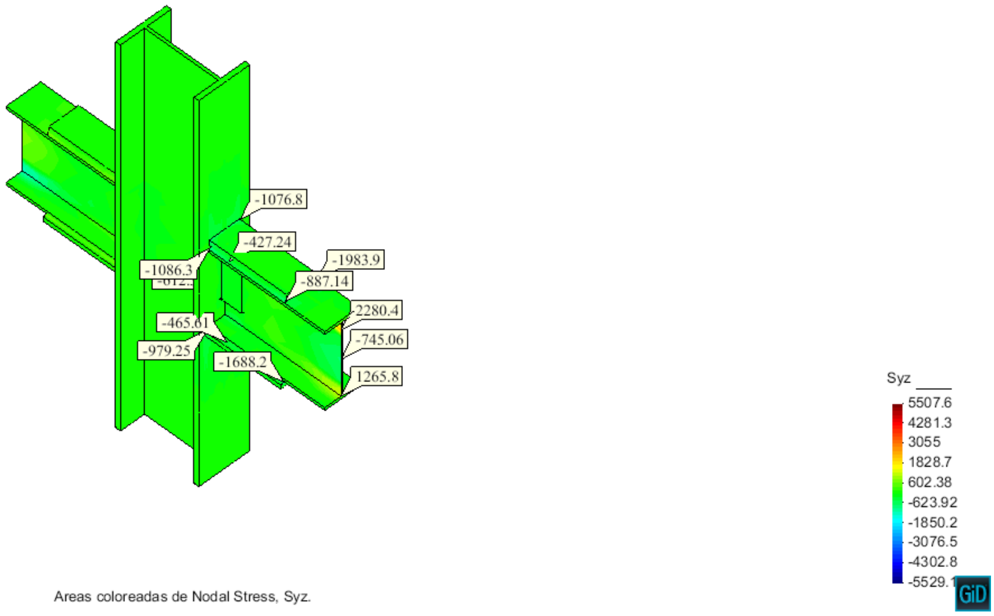


Figure 107: Shear stresses (XY plane, SXY) in M1-A572 conventional model.

Observations:

- All values are found below admissible shear stress of design.

Finite Element Analysis results: Model 1, Shear stresses S_{yz}.



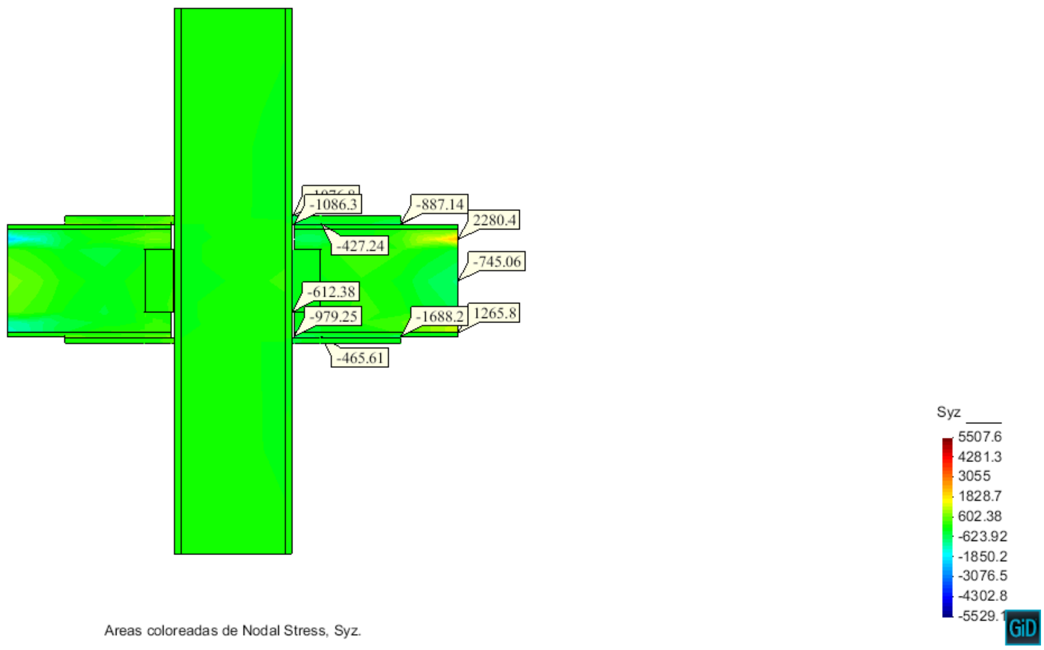


Figure 108: Shear stresses (YZ plane, SYZ) in M1-A572 conventional model.

Observations:

- The extremities of fillet welds connected to bending plates are exceeding shear stress of design (upper 26%, and lower 7.6%).
- The web of the beam have a considerable concentration.

Finite Element Analysis results: Model 1, Von Mises stresses VM_s.

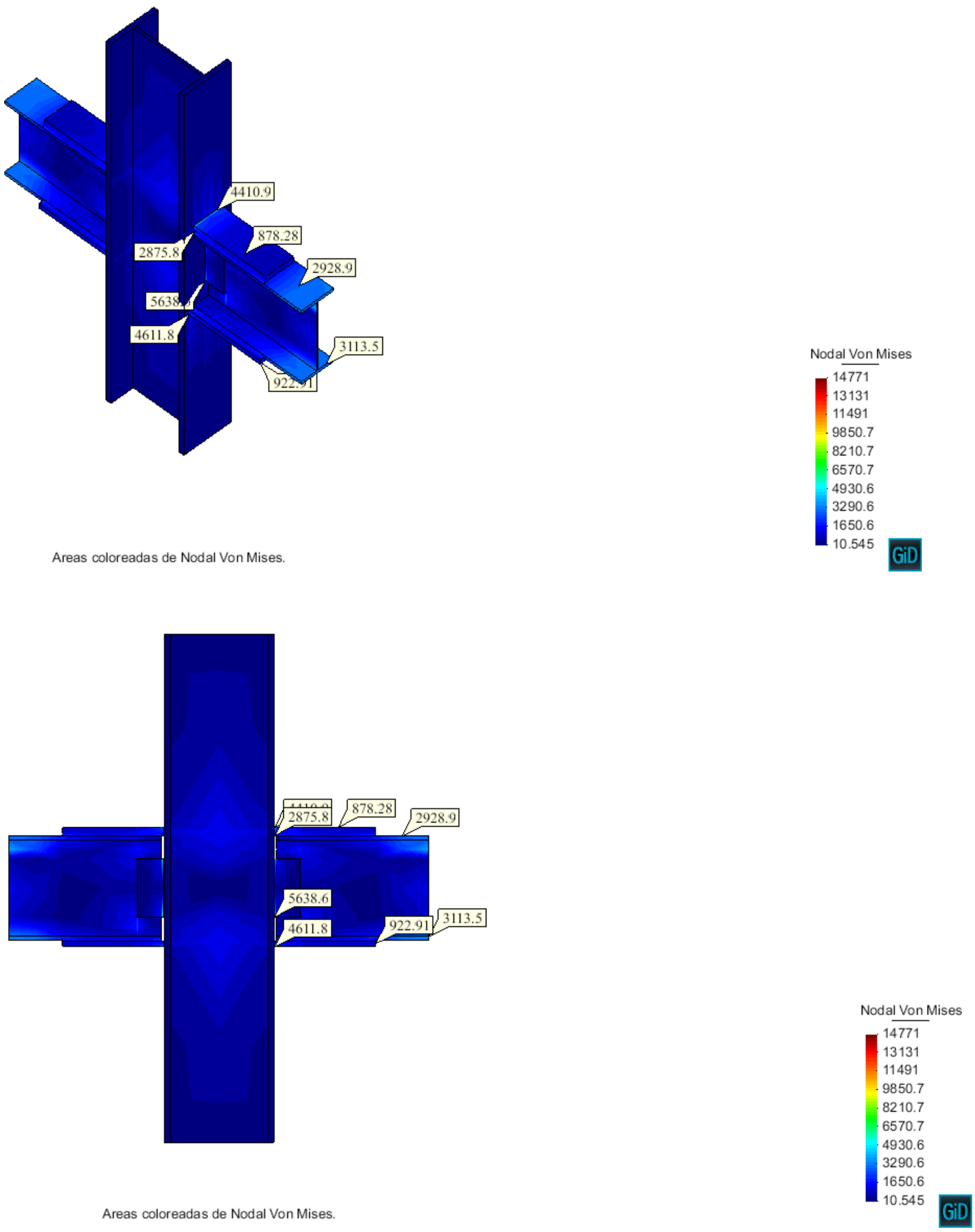


Figure 109: Von Mises stresses in M1-A572 conventional model.

Observations:

- The greater stresses are found very close to rupture parameter.

7.1.3 Model 2.

Input Data

Name of model: M2-BM2-CL2-A572GR50, (model for library building)
Beam: (BM_2) W18x46 — Column: (CL_2) W16x100 — Steel Designation: ASTM-A572 Gr. 50
F_y= $3515 kg/cm^2$ $(50 ksi)$ — F_u= $4570 kg/cm^2$ $(65 ksi)$ — Thickness governant: Column
The flange of beam fits in the flange of column $(0.85 b_{fc} > b_{fb})$.
From beam:

- M_{pf}: $3.76\ e + 06\ kg \cdot cm$ flanges plastic moment contribution.
- M_{pw}: $1.48\ e + 06\ kg \cdot cm$ web plastic moment contribution.
- M_p: $5.24\ e + 06\ kg \cdot cm$ total plastic moment.
- M_y: $4.54\ e + 06\ kg \cdot cm$ yield moment.
- $\frac{M_{pf}}{M_p} = 71.76\%$ percentage contribution of flanges respect to total plastic moment.
- $\frac{M_{pw}}{M_p} = 28.24\%$ percentage contribution of web respect to total plastic moment.

Design forces:

- $V_u = 2.99\ e + 04\ kg$ ultimate shear load.
- $V_s = 2.30\ e + 04\ kg$ service shear load.
- $M_u = 3.34\ e + 06\ kg \cdot cm$ ultimate moment.
- $M_s = 2.57\ e + 06\ kg \cdot cm$ service moment.

Rigid Upper Plate

From beam:

Flexural strength.

$F(kg \cdot cm)$	LRFD (ϕM_n)	ASD $(\frac{M_n}{\Omega})$
Nominal moment	4.71e+06	3.13e+06

$F_{ut} = F_{uc} = 1.14\ e+05\ kg$ ultimate tension/compression force to design bending plates (according to M_u).
$F_{st} = F_{sc} = 0.56\ e + 05\ kg$ service tension/compression force to design bending plates (according to M_s).

Resultant dimensions:

- $b_p = 13.02 cm$ $(5.125 in)$ width of plate.
- $t_p = 3.18 cm$ $(1.125 in)$ thickness of plate.
- $L_p = 40.00 cm$ $(16.00 in)$ length of plate.
- $\psi = 1.15$ shape factor.

Upper plate nominal force.

$F(kg)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal yield force	1.31e+05	0.87e+05
Nominal rupture force	1.42e+05	0.94e+05

Fillet Welds for Rigid Plates

Results:

- $EXX/FX = 4900kg/cm^2\ (70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.80cm$ minimum leg size of weld.
- $tw_{max} = 2.98cm$ maximum leg size of weld.
- $w_p = 1.00cm$ leg size of weld for design of project.
- $Lw_{req} = 73.08cm$ length of weld required.
- $Lw_p = 78.00cm$ length of weld to be adjusted to the project.
- $Lw_{eff} = 78.00cm$ effective length of proportioned weld.
- $R_{nw} = 2078.58kg/cm$ theoretical strength of weld.

Nominal strength of fillet weld.		
$F(kg/cm)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal strength	1558.94	1039.29

Nominal force due to fillet welds.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal force	1.22e+05	0.81e+05

Rigid Lower Plate

Results:

- $b_p = 17.78cm\ (7.00in)$ width of plate.
- $t_p = 2.54cm\ (1.00in)$ thickness of plate.
- $L_p = 40.00cm\ (16.00in)$ length of plate.
- $r = 0.733cm$ gyration radius of plate.
- $k = 0.65$ effective length factor.
- $kL_p = 26.00cm$ effective length.
- $\frac{kL_p}{r} = 35.46$ slenderness ratio (adimensional).
- $F_e = 16.01\ e+03\ kg/cm^2$ Euler stress.
- $F_{cr} = 3.20\ e+03\ kg/cm^2$ critical stress.
- $P_n = 1.45\ e+05\ kg$ theoretical compression force resisted.

Compression force resisted by lower plate.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal force	1.30e+05	0.87e+05

Shear Design

Beam checking:

- $\frac{h}{t_w}$ = 50.28 slenderness ratio of web.
- k_v = 5.34 (a=0, no transverse stiffeners) adimensional value.
- C_{v1} = 1.00 adimensional value.
- $A_w = 42.04cm^2$ area of the web.

Shear strength of beam section.		
$F(kg)$	LRFD (ϕV_n)	ASD ($\frac{V_n}{\Omega}$)
Nominal shear resistance	7.98e+04	5.31e+04

Shear plates:

- $b_p = 10.16cm$ $(4.00in)$ width of plate.
- $t_p = 0.95cm$ $(0.375in)$ thickness of plate.
- L_p *or* $h = 22.86cm$ $(9.00in)$ length of plate.

Strength of shear plates (the pair).		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal yield strength	8.27e+04	5.50e+04
Nominal rupture strength	8.96e+04	5.97e+04

Results for fillet welds of shear plates:

- $EXX/FX = 4900kg/cm^2$ $(70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.50cm$ minimum leg size of weld.
- $tw_{max} = 0.75cm$ maximum leg size of weld.
- $w_p = 0.50cm$ leg size of weld for design of project.
- $Lw_p = 45.72cm$ length of weld proportioned.
- $Lw_{eff} = 45.72cm$ effective length of proportioned weld.

Nominal strength of fillet welds for shear plates.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal strength	3.56e+04	2.38e+04

Additional quantities.		
Weight	*Length of fillet weld (total)*	*Length of groove weld (total)*
30.64 Kg	45.72 cm of 5 mm, and 156.00 cm of 10 mm	30.80 cm

Flanges and Webs with concentrated forces (Chapter J.10)

Flange local bending.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	4.67e+04	3.11e+04
Column	12.37e+04	8.23e+04

From column:

- $t_w = 1.49cm$ thickness of web.
- $k = 3.53cm$ eccentricity of W shape.
- $l_b = 4.50cm$ bearing length (min(t_p,t_{fb})+2w); where:
  - w is the leg size of weld project (w_p).
  - t_p is the thicker thickness of the bending plates.
  - t_{fb} is the thickness of the flange beam.

Web local yielding.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	11.05e+04	7.37e+04

From column:

- $t_f = 2.50cm$ thickness of flange.
- $h = 43.18cm$ depth of W shape.
- $l_b = 4.50cm$ bearing length.
- $Q_f = 1.00$ adimensional factor.

Web local crippling.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	16.64e+04	11.09e+04

Transverse stiffeners (Chapter G2, Section 3 from ANSI/AISC-360-16):

Feature	**Beam**	**Column**
h	45.97cm	43.18cm
t_w	0.91cm	1.49cm
$\frac{h}{t_w}$	50.28	29.06
$2.46\sqrt{\frac{E}{F_{yw}}}$	59.26	59.26
Requirement?	NO	NO

- $F_{yw} = 3515kg/cm^2$ yield stress.
- Transverse stiffeners **not necessary if**:

$$\frac{h}{t_w} < 2.46\sqrt{\frac{E}{F_{yw}}}$$

Stiffening/Continuity Plates

- $C_u = T_u$: 0.00
- A_{req}: $1.08cm^2$
- $\frac{b_{fb}}{3}$: $5.13cm$
- $0.56\sqrt{\frac{E}{F_y}}$: 13.49

For upper plate ($t_{up} = 3.18cm$ $(1.25in)$):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	33.53	34.37	10.56	10.60e+04	7.06e+04
Mid plate	16.76	34.37	5.28	5.80e+04	3.53e+04
Optimal	0.00	34.37	0.00	0.00	0.00

For lower plate ($t_{lp} = 2.54cm$ $(1.00in)$):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	26.82	34.37	10.56	8.48e+04	5.65e+04
Mid plate	13.41	34.37	5.28	4.24e+04	2.83e+04
Optimal	0.00	34.37	0.00	0.00	0.00

Note: strengths (kg) are expressed for just one plate.
For welds (two alternatives, it considers upper and lower edges):

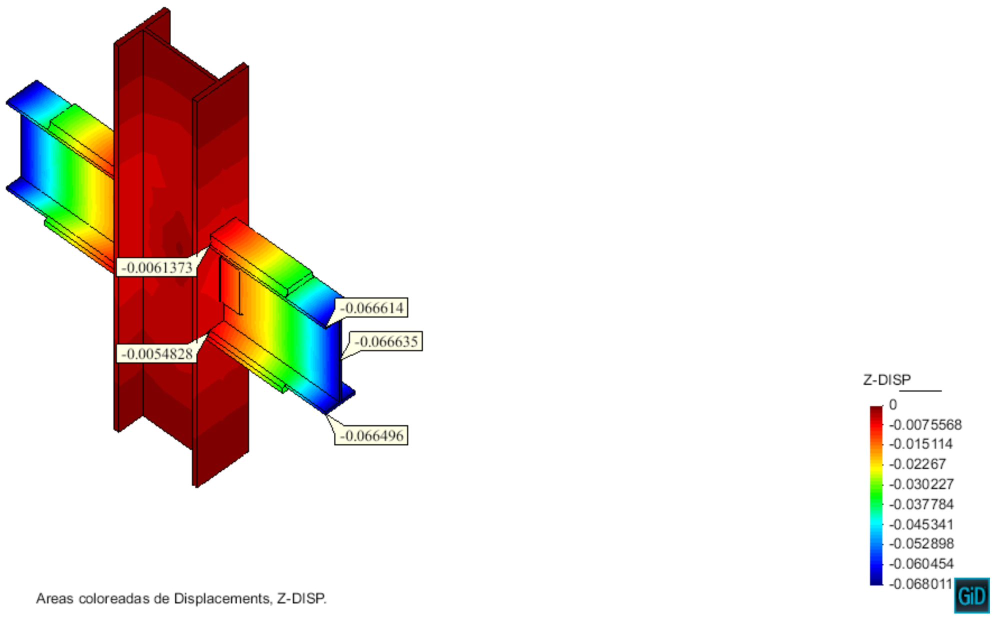
- t_{w1} (weld leg size for alternative 1) $= 0.30cm$ $(\frac{1}{8})''$
- t_{w2} (weld leg size for alternative 2) $= 0.50cm$

Plate	$LRFD(\phi R_{n1})$	$ASD(\frac{R_{n1}}{\Omega})$	$LRFD(\phi R_{n2})$	$ASD(\frac{R_{n2}}{\Omega})$
FP/MP/OP	3.21e+04	2.14e+04	5.36e+04	3.57e+04

Observations

- Bending plates are estimated an equal yield strength.
- Resistance is highly dependant of fillet welds strength.
- The ratio percentage of shear welds designed respect to the features projected is 84% of the total strength.

Finite Element Analysis results: Model 2, Displacements.



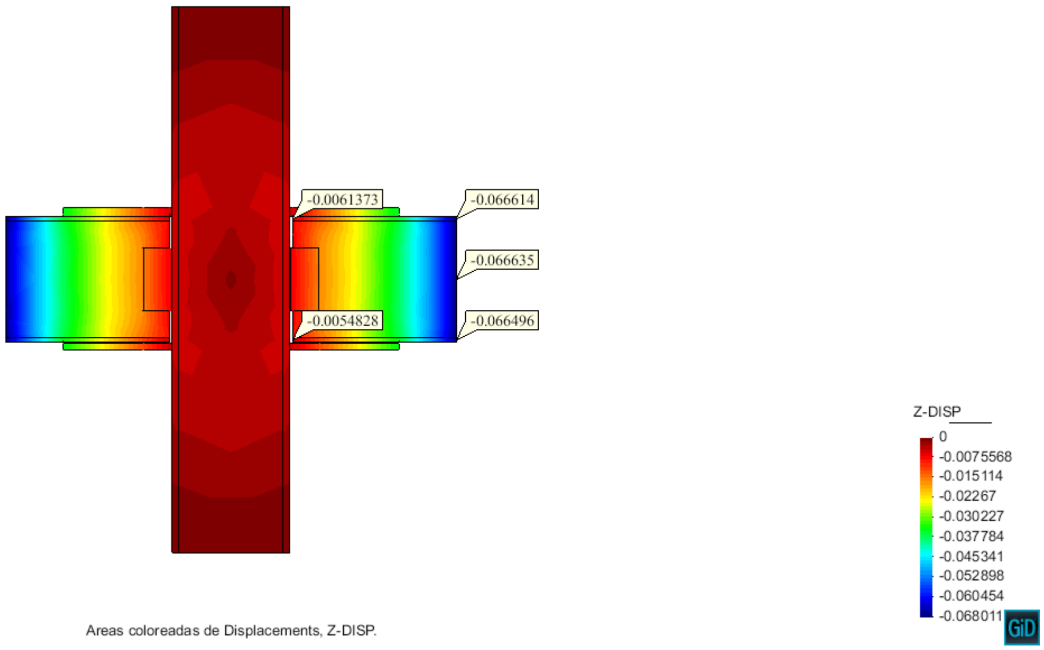


Figure 110: Vertical displacements (Z-axis) in M2-A572 conventional model.

Finite Element Analysis results: Model 2, Tension stresses S_{yy}.

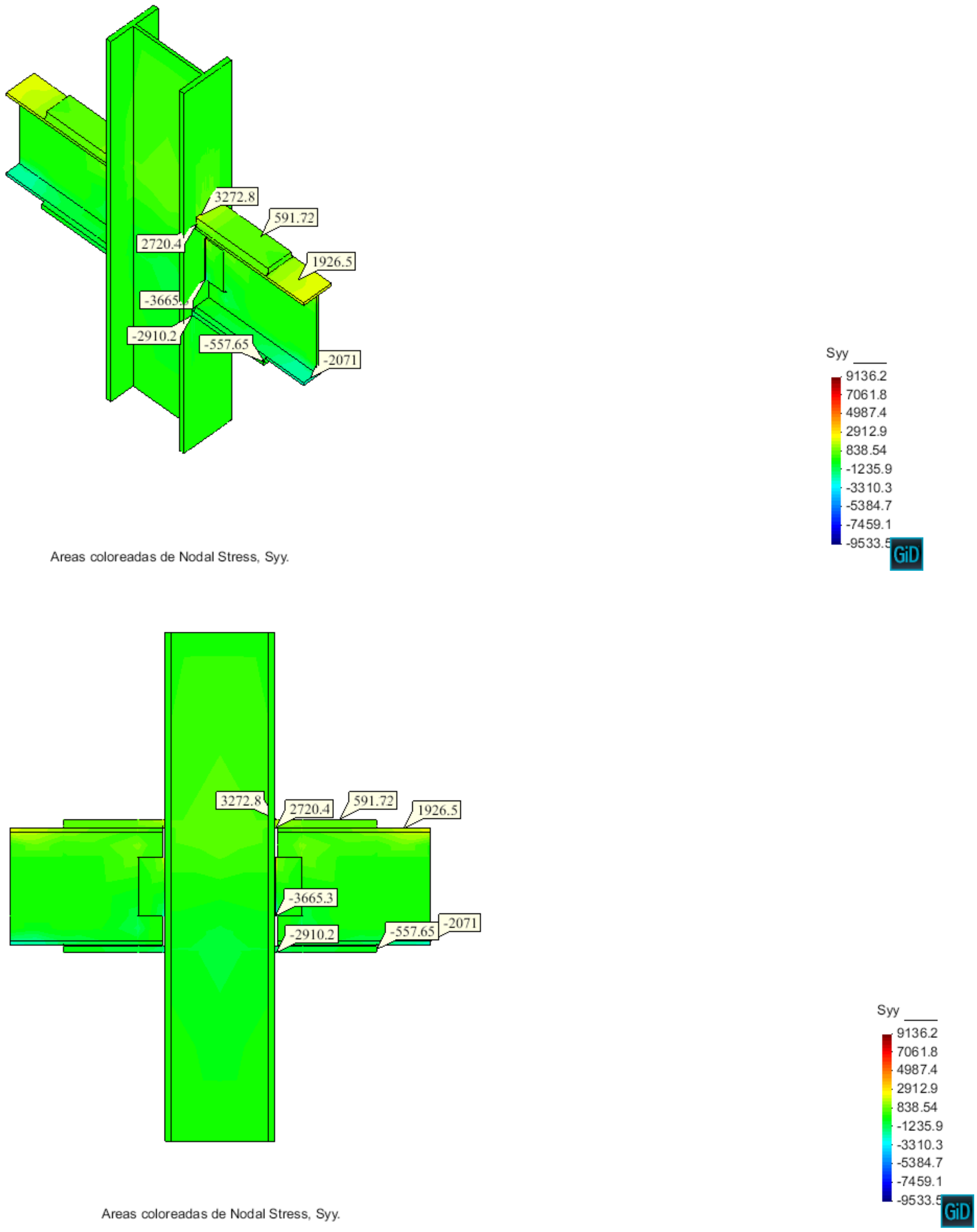


Figure 111: Axial stresses (Y-axis, SYY) in M2-A572 conventional model.

Observations:

- All values are below yield stress.

Finite Element Analysis results: Model 2, Shear stresses S_{xy}.

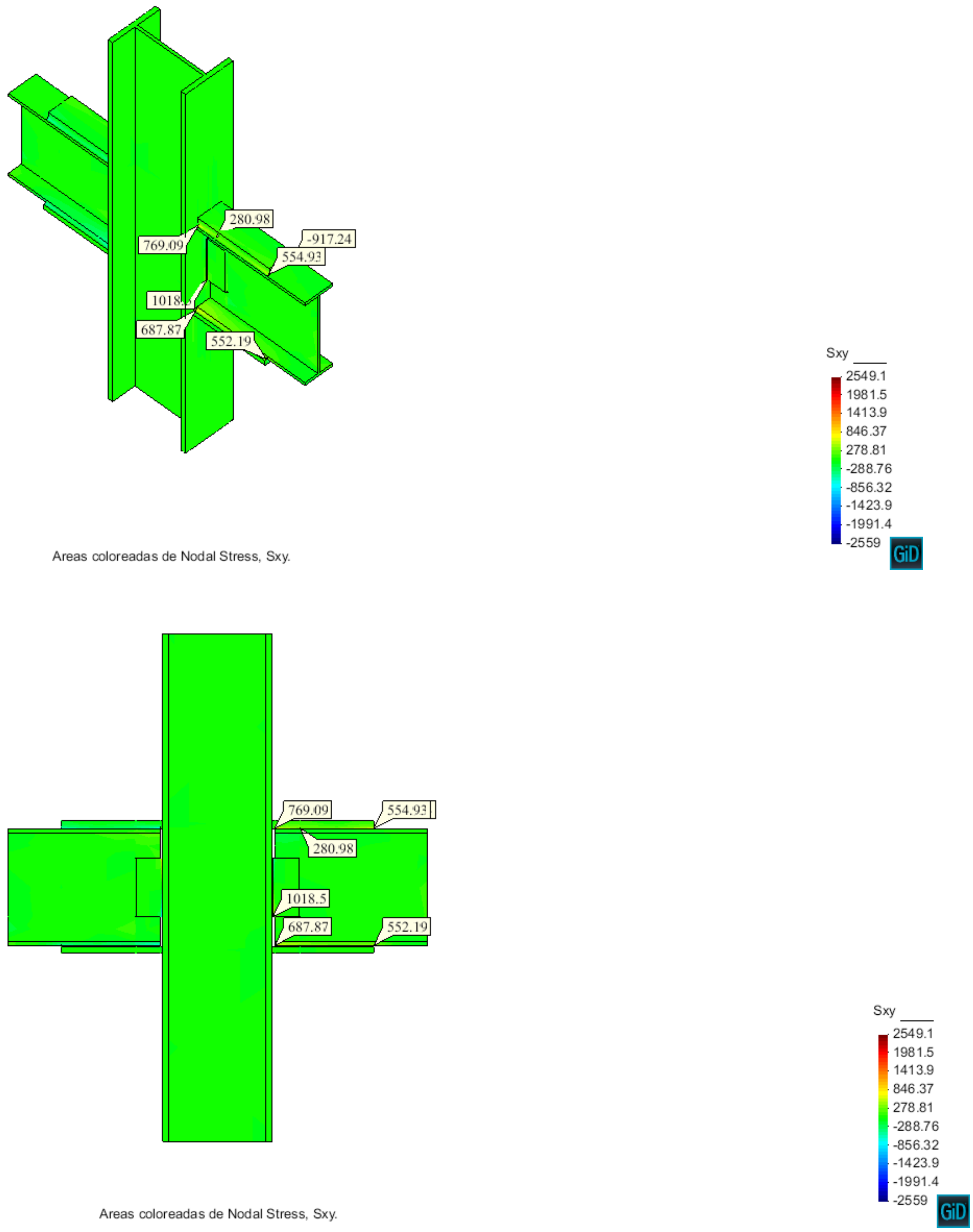
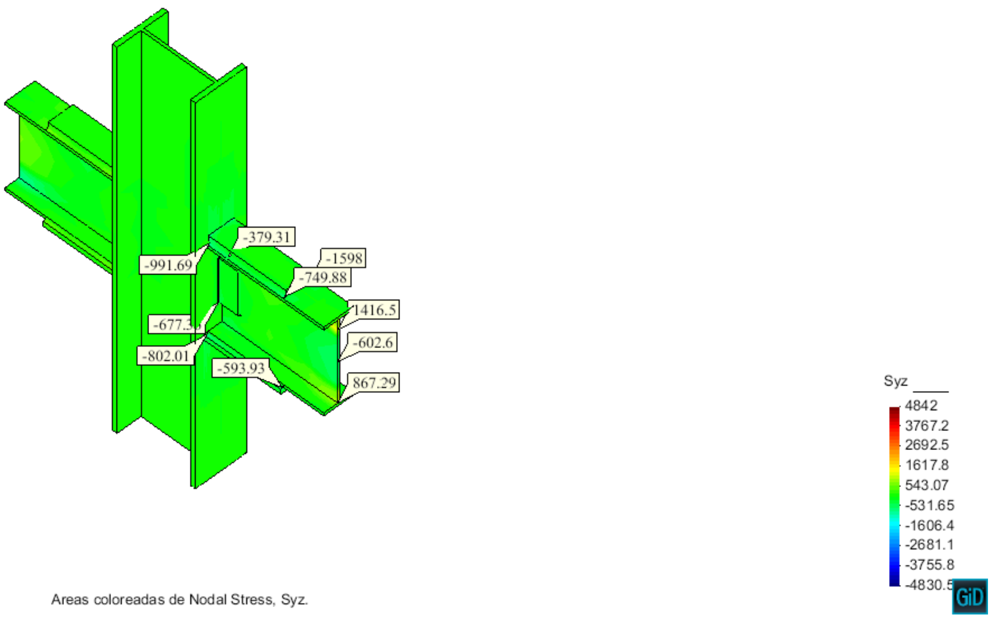


Figure 112: Shear stresses (XY plane, SXY) in M2-A572 conventional model.

Observations:

- Fillet welds have an acceptable performance.

Finite Element Analysis results: Model 2, Shear stresses S_{yz}.



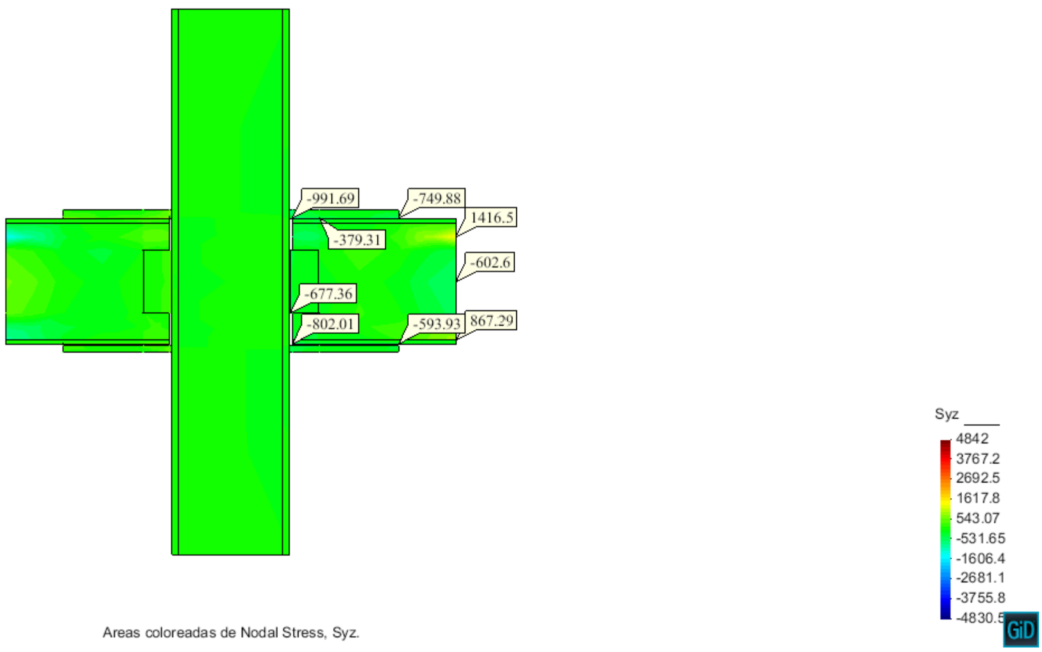


Figure 113: Shear stresses (YZ plane, SYZ) in M2-A572 conventional model.

Observations:

- Fillet welds have an acceptable performance.
- The web of the beam have a notorious reduction of shear stress compared to previous conventional models (0 and 1).

Finite Element Analysis results: Model 2, Von Mises stresses VM_s.

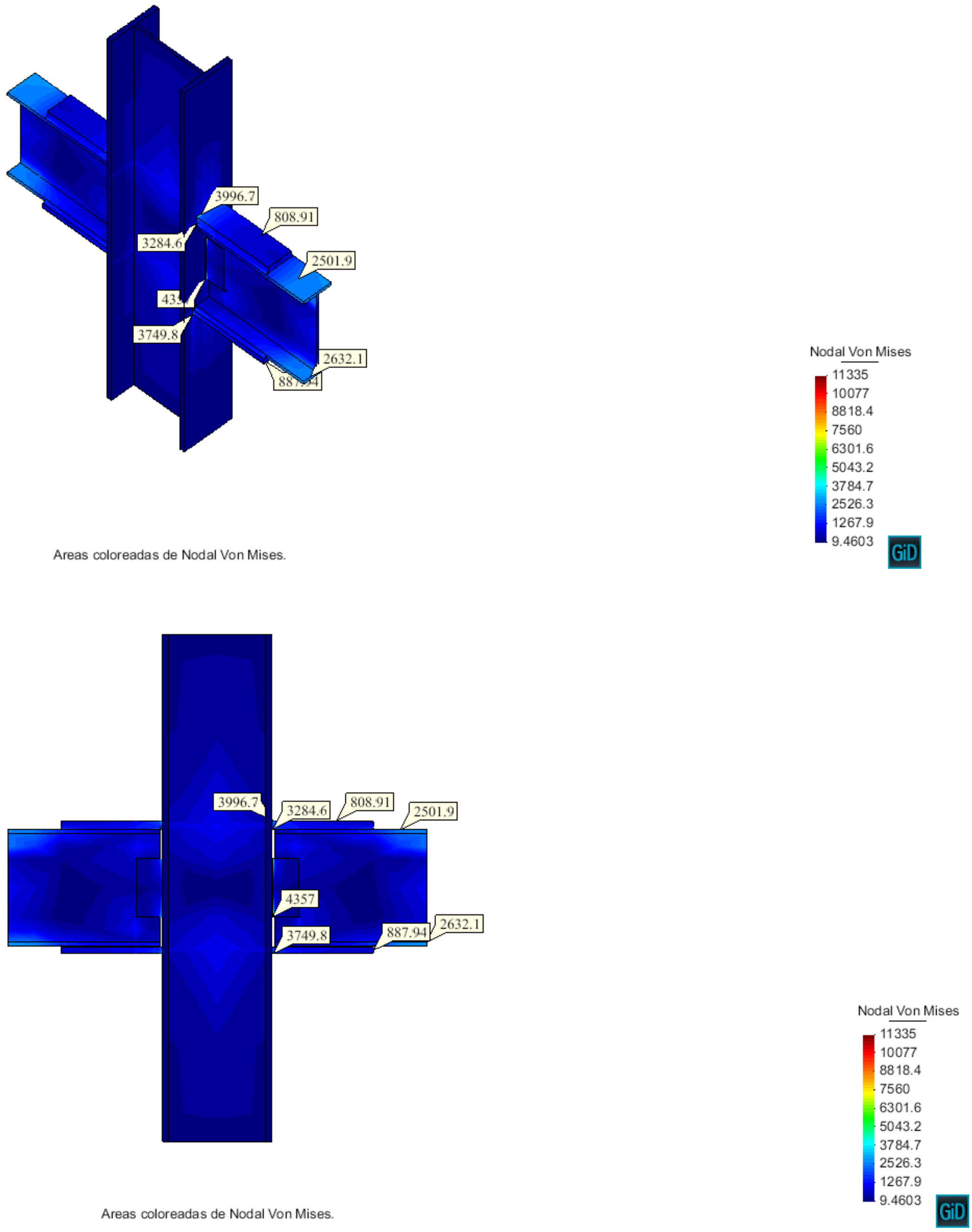


Figure 114: Von Mises stresses in M2-A572 conventional model.

7.1.4 Model 3.

Input Data

Name of model: M3-BM3-CL3-A572GR50, (arbitrary model)
Beam: (BM_3) W24x55 — Column: (CL_3) W16x100 — Steel Designation: ASTM-A572 Gr. 50
F_y= $3515kg/cm^2$ $(50ksi)$ — F_u= $4570kg/cm^2$ $(65ksi)$ — Thickness governant: Column
The flange of beam fits in the flange of column $(0.85b_{fc} > b_{fb})$.
From beam:

- M_{pf}: $4.76\ e+06\ kg \cdot cm$ flanges plastic moment contribution.
- M_{pw}: $2.90\ e+06\ kg \cdot cm$ web plastic moment contribution.
- M_p: $7.66\ e+06\ kg \cdot cm$ total plastic moment.
- M_y: $6.57\ e+06\ kg \cdot cm$ yield moment.
- $\dfrac{M_{pf}}{M_p} = 62.12\%$ percentage contribution of flanges respect to total plastic moment.
- $\dfrac{M_{pw}}{M_p} = 37.88\%$ percentage contribution of web respect to total plastic moment.

Design forces:

- $V_u = 3.77\ e+04\ kg$ ultimate shear load.
- $V_s = 2.69\ e+04\ kg$ service shear load.
- $M_u = 5.25\ e+06\ kg \cdot cm$ ultimate moment.
- $M_s = 3.75\ e+06\ kg \cdot cm$ service moment.

Rigid Upper Plate

From beam:

Flexural strength.		
$F(kg \cdot cm)$	LRFD (ϕM_n)	ASD $(\frac{M_n}{\Omega})$
Nominal moment	6.90e+06	4.59e+06

$F_{ut} = F_{uc} = 1.28\ e+05\ kg$ ultimate tension/compression force to design bending plates (according to M_u).
$F_{st} = F_{sc} = 0.63\ e+05\ kg$ service tension/compression force to design bending plates (according to M_s).

Resultant dimensions:

- $b_p = 15.56cm\ (6.125in)$ width of plate.
- $t_p = 3.18cm\ (1.125in)$ thickness of plate.
- $L_p = 45.00cm\ (18.00in)$ length of plate.
- $\psi = 1.18$ shape factor.

Upper plate nominal force.		
$F(kg)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal yield force	1.56e+05	1.04e+05
Nominal rupture force	1.69e+05	1.13e+05

Fillet Welds for Rigid Plates

Results:

- $EXX/FX = 4900kg/cm^2$ $(70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.80cm$ minimum leg size of weld.
- $tw_{max} = 2.98cm$ maximum leg size of weld.
- $w_p = 1.00cm$ leg size of weld for design of project.
- $Lw_{req} = 82.01cm$ length of weld required.
- $Lw_p = 88.00cm$ length of weld to be adjusted to the project.
- $Lw_{eff} = 88.00cm$ effective length of proportioned weld.
- $R_{nw} = 2078.58kg/cm$ theoretical strength of weld.

Nominal strength of fillet weld.		
$F(kg/cm)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal strength	1558.94	1039.29

Nominal force due to fillet welds.		
$F(kg)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal force	1.37e+05	0.91e+05

Rigid Lower Plate

Results:

- $b_p = 21.59cm$ $(8.50in)$ width of plate.
- $t_p = 2.54cm$ $(1.00in)$ thickness of plate.
- $L_p = 45.00cm$ $(18.00in)$ length of plate.
- $r = 0.733cm$ gyration radius of plate.
- $k = 0.65$ effective length factor.
- $kL_p = 29.25cm$ effective length.
- $\frac{kL_p}{r} = 39.89$ slenderness ratio (adimensional).
- $F_e = 12.65\ e + 03\ kg/cm^2$ Euler stress.
- $F_{cr} = 3.13\ e + 03\ kg/cm^2$ critical stress.
- $P_n = 1.72\ e + 05\ kg$ theoretical compression force resisted.

Compression force resisted by lower plate.		
$F(kg)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal force	1.54e+05	1.03e+05

Shear Design

Beam checking:

- $\dfrac{h}{t_w}$ = 59.75 slenderness ratio of web.
- k_v = 5.34 (a=0, no transverse stiffeners) adimensional value.
- C_{v1} = 1.00 adimensional value.
- $A_w = 60.14 cm^2$ area of the web.

Shear strength of beam section.		
$F(kg)$	LRFD (ϕV_n)	ASD ($\frac{V_n}{\Omega}$)
Nominal shear resistance	11.42e+04	7.59e+04

Shear plates:

- $b_p = 10.16cm$ $(4.00in)$ width of plate.
- $t_p = 0.95cm$ $(0.375in)$ thickness of plate.
- L_p or $h = 35.56cm$ $(14.00in)$ length of plate.

Strength of shear plates (the pair).		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal yield strength	12.86e+04	8.55e+04
Nominal rupture strength	13.93e+04	9.29e+04

Results for fillet welds of shear plates:

- $EXX/FX = 4900kg/cm^2$ $(70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.50cm$ minimum leg size of weld.
- $tw_{max} = 0.75cm$ maximum leg size of weld.
- $w_p = 0.625cm$ leg size of weld for design of project.
- $Lw_p = 71.12cm$ length of weld proportioned.
- $Lw_{eff} = 69.16cm$ effective length of proportioned weld.

Nominal strength of fillet welds for shear plates.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal strength	6.74e+04	4.49e+04

Additional quantities.		
Weight	*Length of fillet weld (total)*	*Length of groove weld (total)*
42.24 Kg	71.12 cm of 6.3 mm, 176.00 cm of 10 mm	37.15 cm

Flanges and Webs with concentrated forces (Chapter J.10)

Flange local bending.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	3.25e+04	2.16e+04
Column	12.37e+04	8.23e+04

From column:

- $t_w = 1.49cm$ thickness of web.
- $k = 3.53cm$ eccentricity of W shape.
- $l_b = 4.50cm$ bearing length (min(t_p,t_{fb})+2w); where:
  - w is the leg size of weld project (w_p).
  - t_p is the thicker thickness of the bending plates.
  - t_{fb} is the thickness of the flange beam.

Web local yielding.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	11.05e+04	7.37e+04

From column:

- $t_f = 2.50cm$ thickness of flange.
- $h = 43.18cm$ depth of W shape.
- $l_b = 4.50cm$ bearing length.
- $Q_f = 1.00$ adimensional factor.

Web local crippling.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	16.64e+04	11.09e+04

Transverse stiffeners (Chapter G2, Section 3 from ANSI/AISC-360-16):

Feature	**Beam**	**Column**
h	59.94cm	43.18cm
t_w	0.91cm	1.49cm
$\frac{h}{t_w}$	59.75	29.06
$2.46\sqrt{\frac{E}{F_{yw}}}$	59.26	59.26
Requirement?	YES	NO

- $F_{yw} = 3515kg/cm^2$ yield stress.
- Transverse stiffeners **not necessary if**:

$$\frac{h}{t_w} < 2.46\sqrt{\frac{E}{F_{yw}}}$$

Stiffening/Continuity Plates

- $C_u = T_u$: 0.00
- A_{req}: $5.49cm^2$
- $\frac{b_{fb}}{3}$: $5.94cm$
- $0.56\sqrt{\frac{E}{F_y}}$: 13.49

For upper plate ($t_{up} = 3.18cm$ $(1.25in)$):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	33.53	34.37	10.56	10.60e+04	7.06e+04
Mid plate	16.76	34.37	5.28	5.80e+04	3.53e+04
Optimal	6.35	34.37	2.00	2.00e+04	1.34e+04

For lower plate ($t_{lp} = 2.54cm$ $(1.00in)$):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	26.82	34.37	10.56	8.48e+04	5.65e+04
Mid plate	13.41	34.37	5.28	4.24e+04	2.83e+04
Optimal	7.62	34.37	3.00	2.41e+04	1.60e+04

Note: strengths (kg) are expressed for just one plate.
For welds (two alternatives, it considers upper and lower edges):

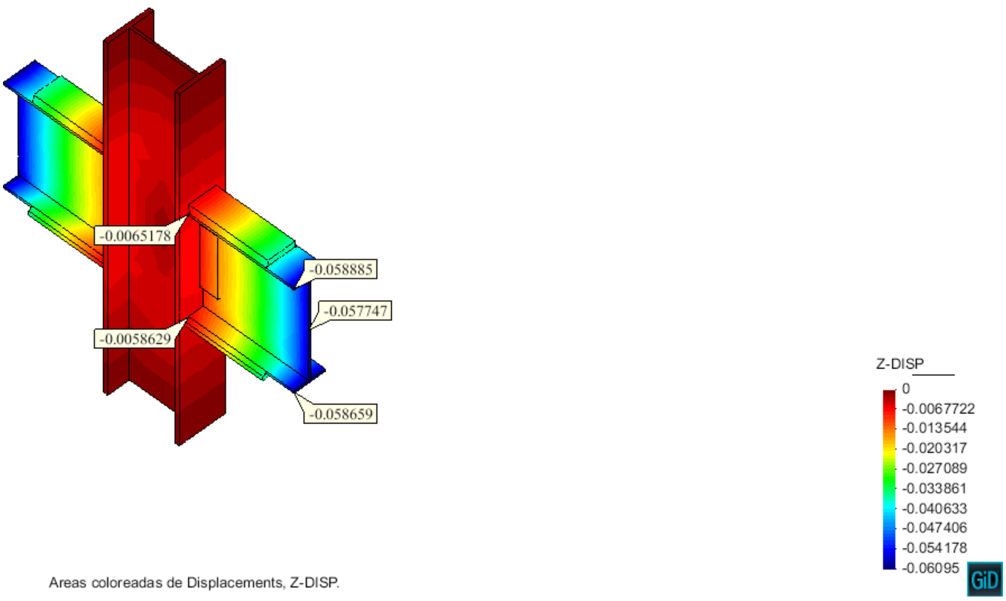
- t_{w1} (weld leg size for alternative 1) = $0.30cm$ $(\frac{1}{8})''$
- t_{w2} (weld leg size for alternative 2) = $0.50cm$

Plate	$LRFD(\phi R_{n1})$	$ASD(\frac{R_{n1}}{\Omega})$	$LRFD(\phi R_{n2})$	$ASD(\frac{R_{n2}}{\Omega})$
FP/MP/OP	3.21e+04	2.14e+04	5.36e+04	3.57e+04

Observations

- Compression plate reaches a 75% of its capacity, and about 81% for the tension plate. Both plates have an almost equal yield strength.
- The fillet welds for shear have a demand estimated up to 55%.
- Structural safety depends on fillet welds strength.

Finite Element Analysis results: Model 3, Displacements.



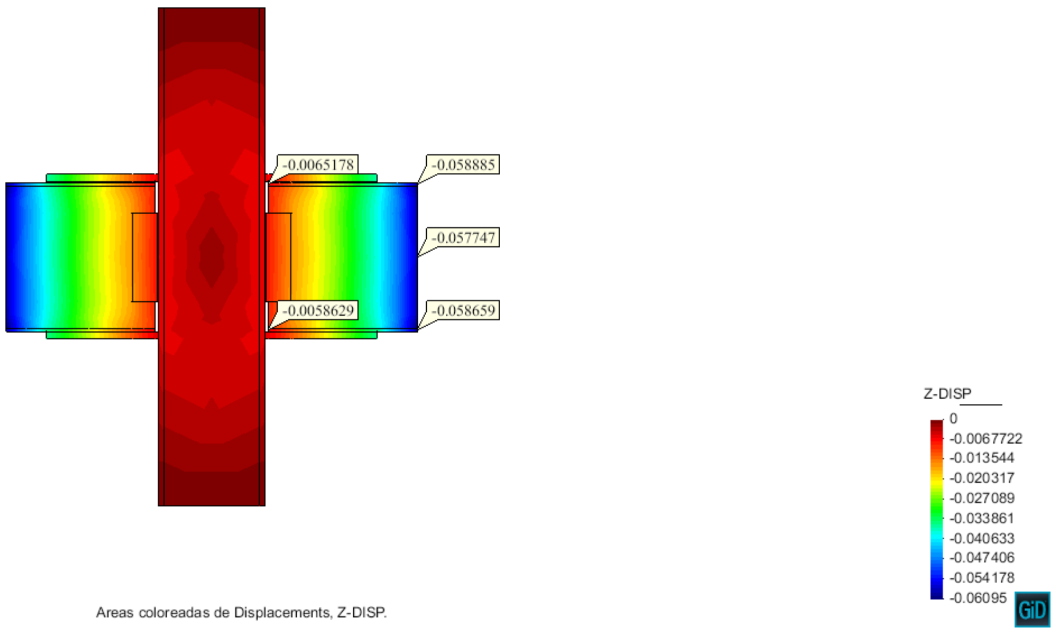
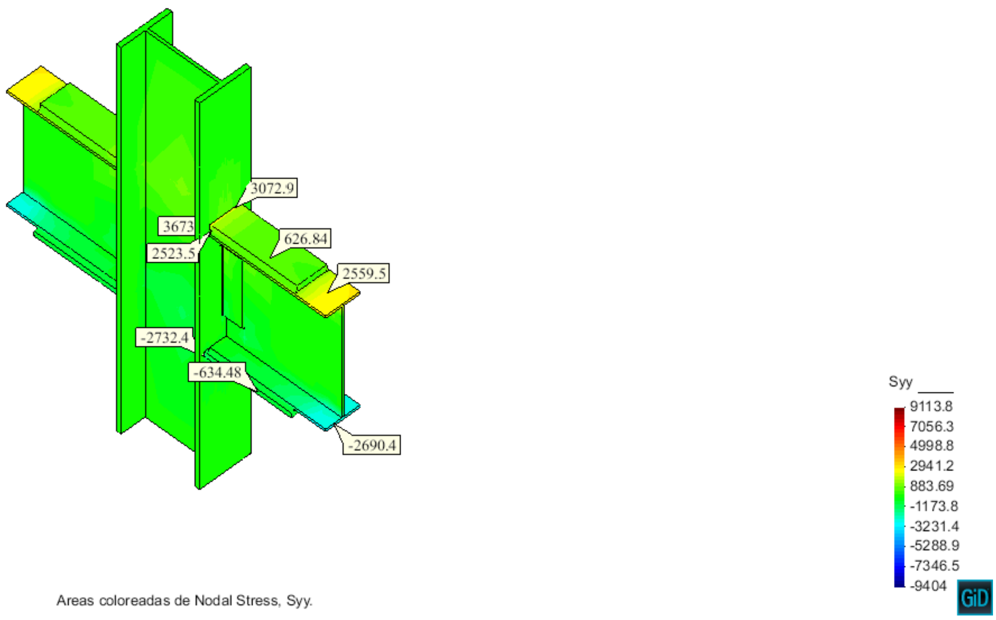


Figure 115: Vertical displacements (Z-axis) in M3-A572 conventional model.

Finite Element Analysis results: Model 3, Tension stresses S_{yy}.



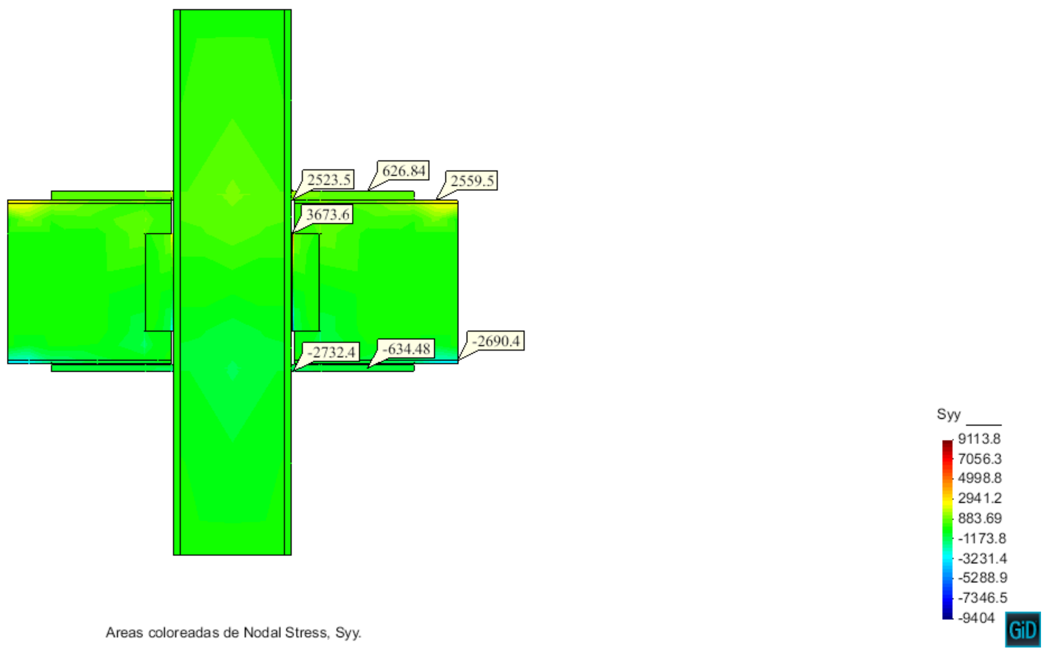


Figure 116: Axial stresses (Y-axis, SYY) in M3-A572 conventional model.

Observations:

- Bending plates have an acceptable development trough flexure.

Finite Element Analysis results: Model 3, Shear stresses S_{xy}.

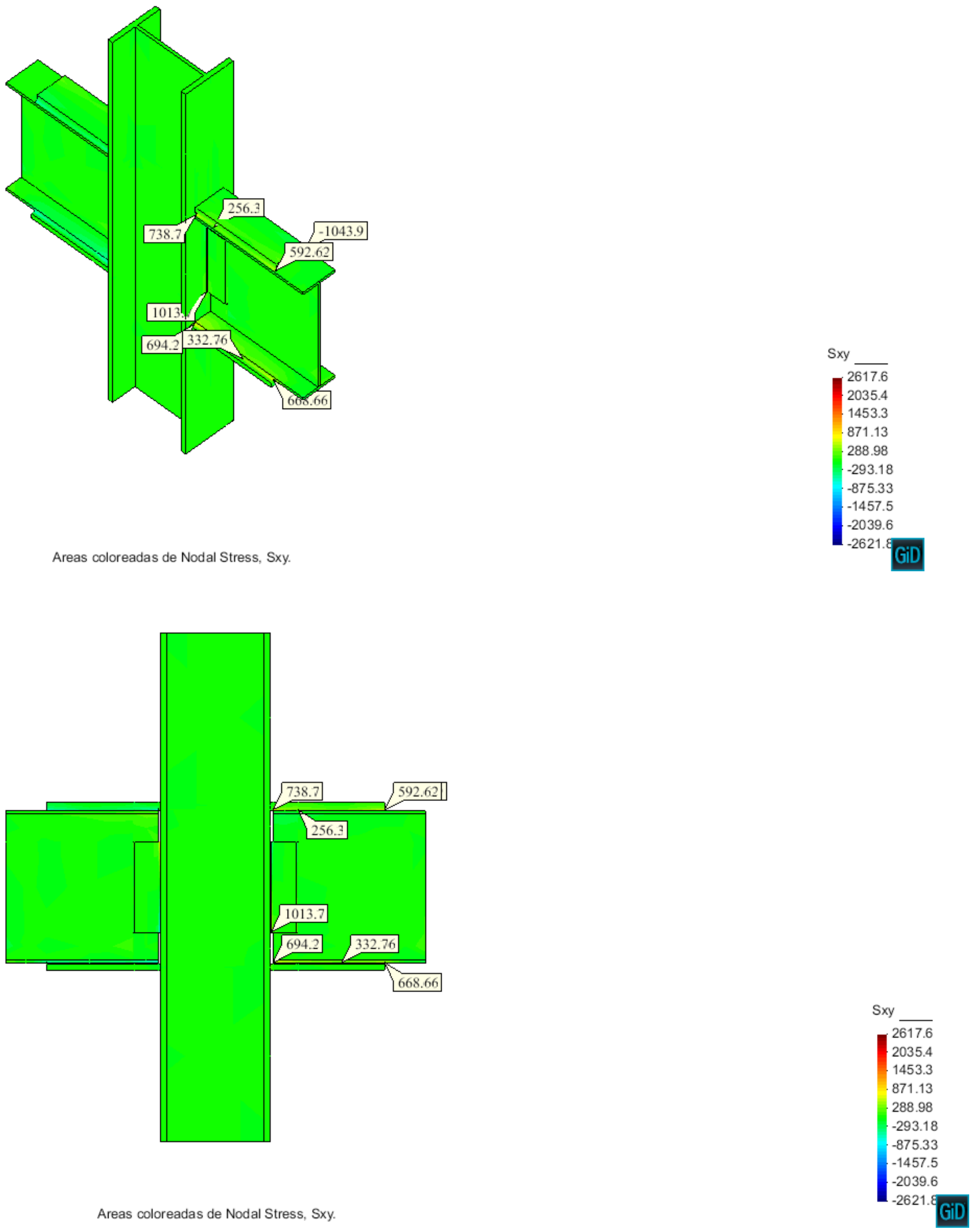
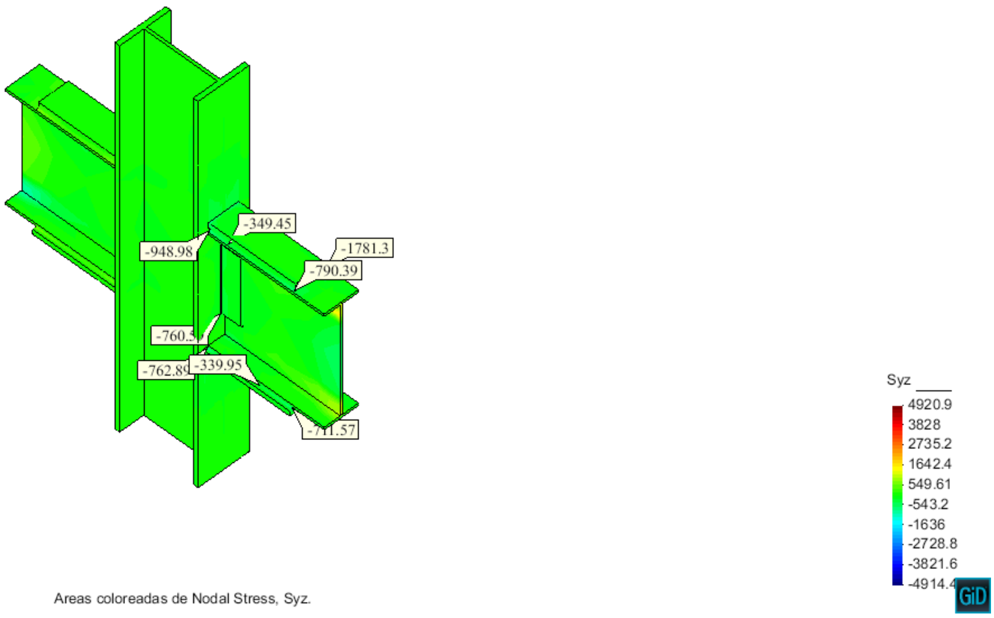


Figure 117: Shear stresses (XY plane, SXY) in M3-A572 conventional model.

Observations:

- All values are located under shear stress design.

Finite Element Analysis results: Model 3, Shear stresses S_{yz}.



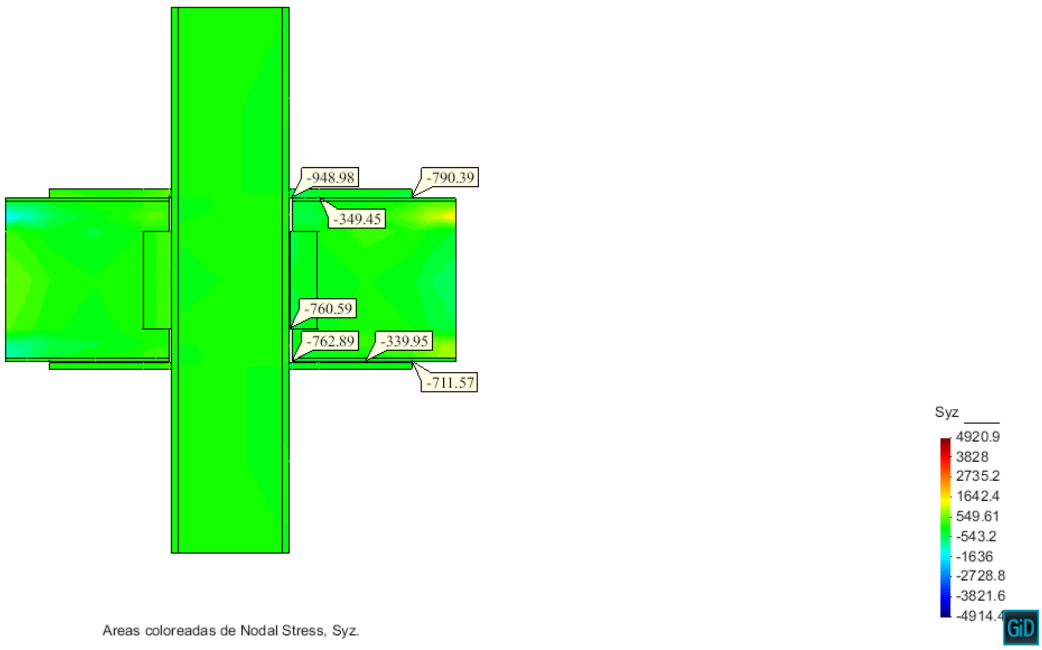


Figure 118: Shear stresses (YZ plane, SYZ) in M3-A572 conventional model.

Observations:

- The upper fillet welds gather a certain concentration in the gaps between flange of the beam and plates. In extremities, the contour values show an excess around 13%.

Finite Element Analysis results: Model 3, Von Mises stresses VM_s.

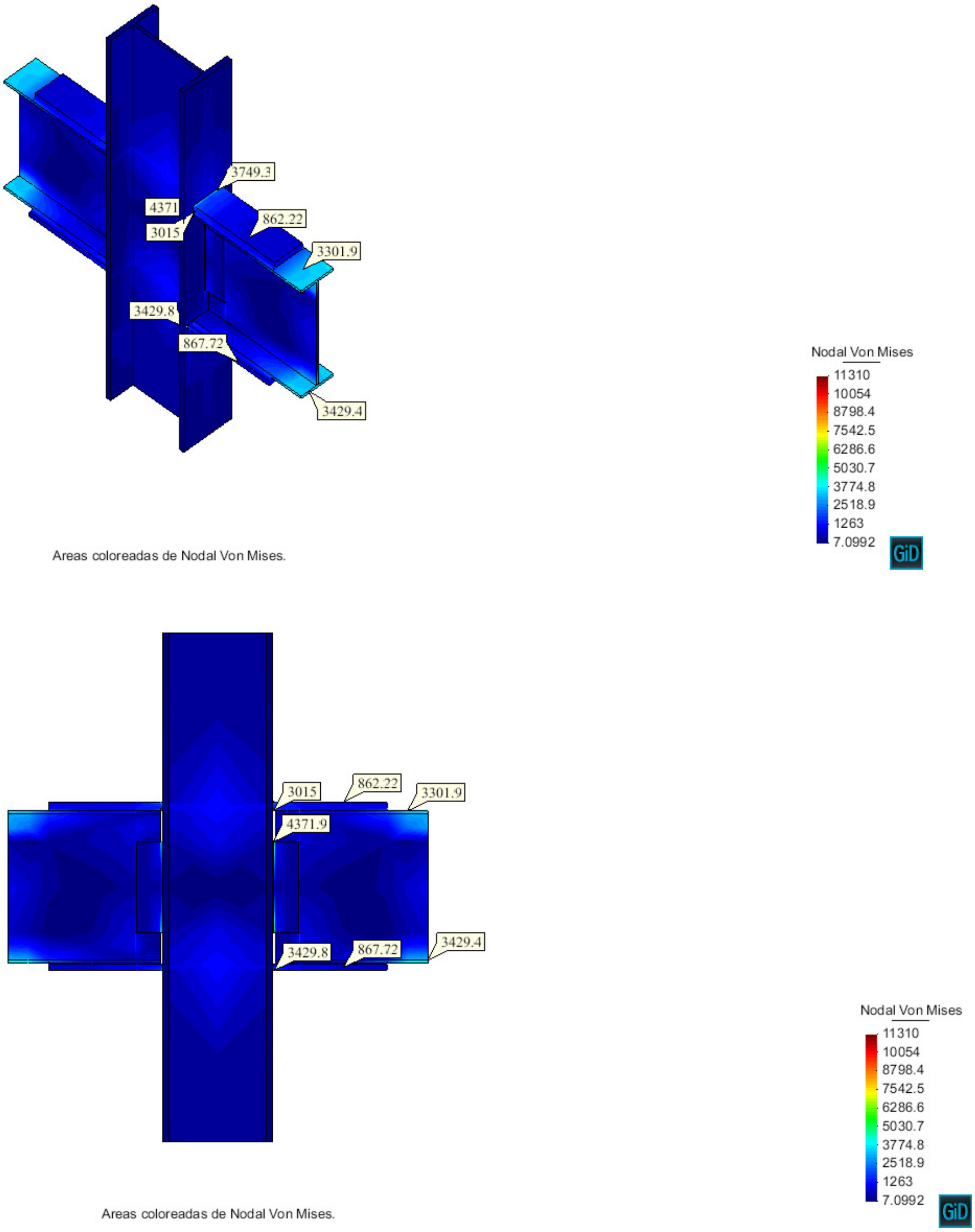


Figure 119: Von Mises stresses in M3-A572 conventional model.

Observation:

- Yield surfaces start to extend including the flanges and partial area of the web.
- Making a comparison for bending plates, just the lower ones get values under yield stress.

7.2 Non-qualified Bending Moment Configurations

7.2.1 Model 0.

Input Data

Name of model: M0-BM0-CL0-A572GR50, (arbitrary model)
Beam: (BM_0) W12x30 — Column: (CL_0) W16x100 — Steel Designation: ASTM-A572 Gr. 50
F_y= $3515 kg/cm^2$ ($50 ksi$) — F_u= $4570 kg/cm^2$ ($65 ksi$) — Thickness governant: Column
The flange of beam fits in the flange of column ($0.85 b_{fc} > b_{fb}$).
From beam:

- M_{pf}: $1.99\ e+06\ kg \cdot cm$ flanges plastic moment contribution.
- M_{pw}: $0.49\ e+06\ kg \cdot cm$ web plastic moment contribution.
- M_p: $2.48\ e+06\ kg \cdot cm$ total plastic moment.
- M_y: $2.22\ e+06\ kg \cdot cm$ yield moment.
- $\frac{M_{pf}}{M_p} = 80.35\%$ percentage contribution of flanges respect to total plastic moment.
- $\frac{M_{pw}}{M_p} = 19.65\%$ percentage contribution of web respect to total plastic moment.

Design forces:

- $V_u = 1.29\ e+04\ kg$ ultimate shear load.
- $V_s = 0.92\ e+04\ kg$ service shear load.
- $M_u = 1.78\ e+06\ kg \cdot cm$ ultimate moment.
- $M_s = 1.27\ e+06\ kg \cdot cm$ service moment.

Rigid Upper Plate

From beam:

Flexural strength.		
$F(kg \cdot cm)$	LRFD (ϕM_n)	ASD ($\frac{M_n}{\Omega}$)
Nominal moment	2.23e+06	1.49e+06

$F_{ut} = F_{uc} = 0.64\ e+05\ kg$ ultimate tension/compression force to design bending plates (according to M_u).
$F_{st} = F_{sc} = 0.41\ e+05\ kg$ service tension/compression force to design bending plates (according to M_s).

Resultant dimensions:

- $b_p = 14.29 cm$ ($5.625 in$) width of plate.
- $t_p = 1.91 cm$ ($0.75 in$) thickness of plate.
- $L_p = 25.00 cm$ ($10.00 in$) length of plate.
- $\psi = 1.12$ shape factor.

Upper plate nominal force.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal yield force	0.86e+05	0.57e+05
Nominal rupture force	0.93e+05	0.62e+05

Fillet Welds for Rigid Plates

Results:

- $EXX/FX = 4900kg/cm^2$ $(70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.80cm$ minimum leg size of weld.
- $tw_{max} = 1.71cm$ maximum leg size of weld.
- $w_p = 1.00cm$ leg size of weld for design of project.
- $Lw_{req} = 40.99cm$ length of weld required.
- $Lw_p = 48.00cm$ length of weld to be adjusted to the project.
- $Lw_{eff} = 48.00cm$ effective length of proportioned weld.
- $R_{nw} = 2078.58kg/cm$ theoretical strength of weld.

Nominal strength of fillet weld.		
$F(kg/cm)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal strength	1558.94	1039.29

Nominal force due to fillet welds.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal force	0.75e+05	0.50e+05

Rigid Lower Plate

Results:

- $b_p = 21.59cm$ $(8.50in)$ width of plate.
- $t_p = 1.91cm$ $(0.75in)$ thickness of plate.
- $L_p = 25.00cm$ $(10.00in)$ length of plate.
- $r = 0.55cm$ gyration radius of plate.
- $k = 0.65$ effective length factor.
- $kL_p = 16.25cm$ effective length.
- $\frac{kL_p}{r} = 29.55$ slenderness ratio (adimensional).
- $F_e = 23.06\ e+03\ kg/cm^2$ Euler stress.
- $F_{cr} = 3.30\ e+03\ kg/cm^2$ critical stress.
- $P_n = 1.36\ e+05\ kg$ theoretical compression force resisted.

Compression force resisted by lower plate.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal force	1.22e+05	0.81e+05

Shear Design

Beam checking:

- $\frac{h}{t_w}$ = 47.31 slenderness ratio of web.
- k_v = 5.34 (a=0, no transverse stiffeners) adimensional value.
- C_{v1} = 1.00 adimensional value.
- $A_w = 20.63cm^2$ area of the web.

Shear strength of beam section.		
$F(kg)$	LRFD (ϕV_n)	ASD ($\frac{V_n}{\Omega}$)
Nominal shear resistance	3.92e+04	2.60e+04

Angles:

- Angle: $L3\ x\ 2\ x\ \frac{3}{8}$
- $b_p = 5.08cm$ $(2.00in)$ width of plate.
- $t_p = 0.95cm$ $(0.375in)$ thickness of plate.
- L_p *or* $h = 17.78cm$ $(7.00in)$ length of plate.

Strength of "back-to-back" angles (the pair).		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal yield strength	6.43e+04	4.28e+04
Nominal rupture strength	6.97e+04	4.64e+04

Results for fillet welds of angles:

- $EXX/FX = 4900kg/cm^2$ $(70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.50cm$ minimum leg size of weld.
- $tw_{max} = 0.75cm$ maximum leg size of weld.
- $w_p = 0.50cm$ leg size of weld for design of project.
- $Lw_p = 35.56cm$ length of weld proportioned.
- $Lw_{eff} = 35.56cm$ effective length of proportioned weld.

Nominal strength of fillet welds for angles.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal strength	2.22e+04	1.48e+04

Additional quantities.		
Weight	*Length of fillet weld (total)*	*Length of groove weld (total)*
16.56 Kg	71.12 cm of 5 mm, and 96.00 cm of 10 mm	35.88 cm

Flanges and Webs with concentrated forces (Chapter J.10)

Flange local bending.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	2.47e+04	1.64e+04
Column	12.37e+04	8.23e+04

From column:

- $t_w = 1.49cm$ thickness of web.
- $k = 3.53cm$ eccentricity of W shape.
- $l_b = 3.91cm$ bearing length (min(t_p,t_{fb})+2w); where:
  - w is the leg size of weld project (w_p).
  - t_p is the thicker thickness of the bending plates.
  - t_{fb} is the thickness of the flange beam.

Web local yielding.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	10.74e+04	7.16e+04

From column:

- $t_f = 2.50cm$ thickness of flange.
- $h = 43.18cm$ depth of W shape.
- $l_b = 3.91cm$ bearing length.
- $Q_f = 1.00$ adimensional factor.

Web local crippling.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	16.36e+04	10.91e+04

Transverse stiffeners (Chapter G2, Section 3 from ANSI/AISC-360-16):

Feature	**Beam**	**Column**
h	31.24cm	43.18cm
t_w	0.66cm	1.49cm
$\frac{h}{t_w}$	47.31	29.06
$2.46\sqrt{\frac{E}{F_{yw}}}$	59.26	59.26
Requirement?	NO	NO

- $F_{yw} = 3515kg/cm^2$ yield stress.
- Transverse stiffeners **not necessary if**:

$$\frac{h}{t_w} < 2.46\sqrt{\frac{E}{F_{yw}}}$$

Stiffening/Continuity Plates

- $C_u = T_u$: 0.00
- A_{req}: 0.00
- $\dfrac{b_{fb}}{3}$: $5.52cm$
- $0.56\sqrt{\dfrac{E}{F_y}}$: 13.49

For upper plate ($t_{up} = 1.91cm$ $(0.75in)$):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	20.12	34.37	10.56	6.37e+04	4.24e+04
Mid plate	10.06	34.37	5.28	3.19e+04	2.12e+04
Optimal	0.00	34.37	0.00	0.00	0.00

For lower plate ($t_{lp} = 1.91cm$ $(0.75in)$):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	20.12	34.37	10.56	6.37e+04	4.24e+04
Mid plate	10.06	34.37	5.28	3.19e+04	2.12e+04
Optimal	0.00	34.37	0.00	0.00	0.00

Note: strengths (kg) are expressed for just one plate.
For welds (two alternatives, it considers upper and lower edges):

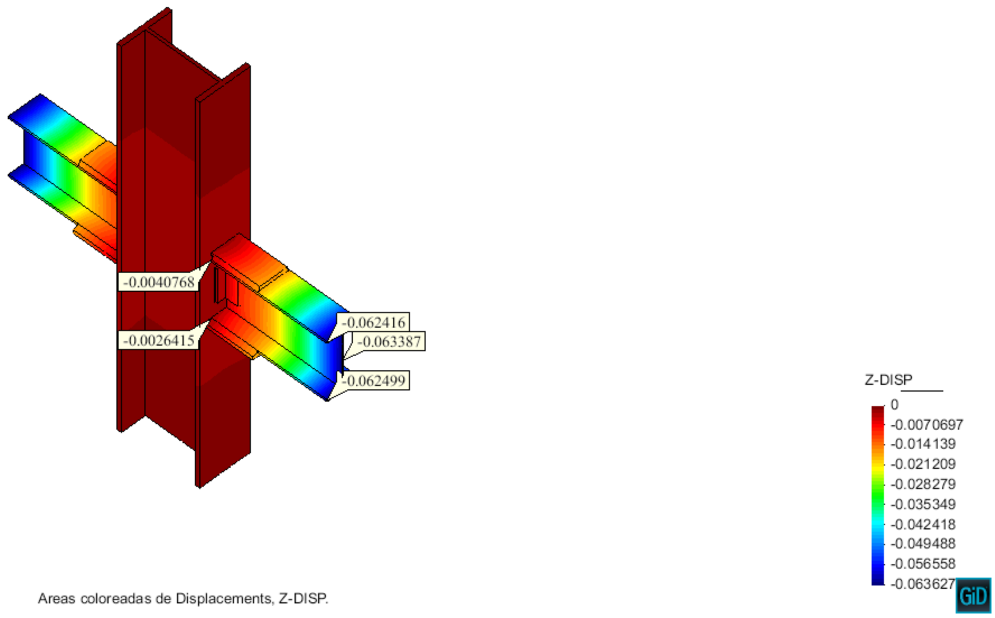
- t_{w1} (weld leg size for alternative 1) $= 0.30cm$ $(\frac{1}{8})''$
- t_{w2} (weld leg size for alternative 2) $= 0.50cm$

Plate	$LRFD(\phi R_{n1})$	$ASD(\frac{R_{n1}}{\Omega})$	$LRFD(\phi R_{n2})$	$ASD(\frac{R_{n2}}{\Omega})$
FP/MP/OP	3.21e+04	2.14e+04	5.36e+04	3.57e+04

Observations

- The lower plate works under the fifty percent, and the upper plate around of 75%.
- The ultimate shear takes a 58% of weld strength.
- Strength is delimited by welds.
- The fillets welds for the short legs projected to be welded on the column flanges can not resist the design stresses.
- Stiffening plates are not required.

Finite Element Analysis results: Model 0, Displacements.



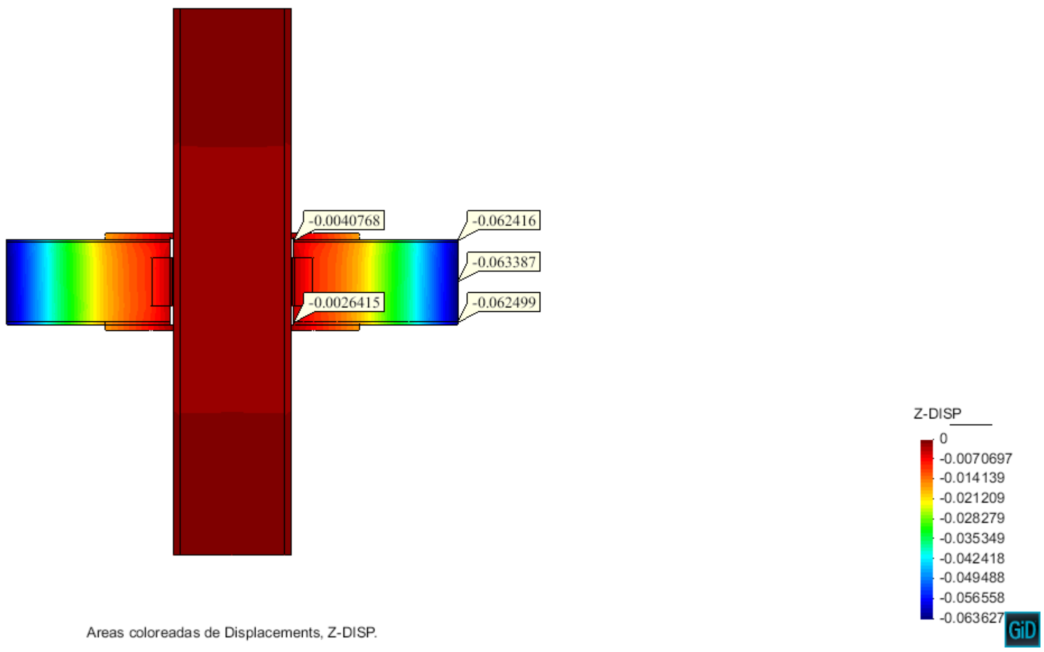
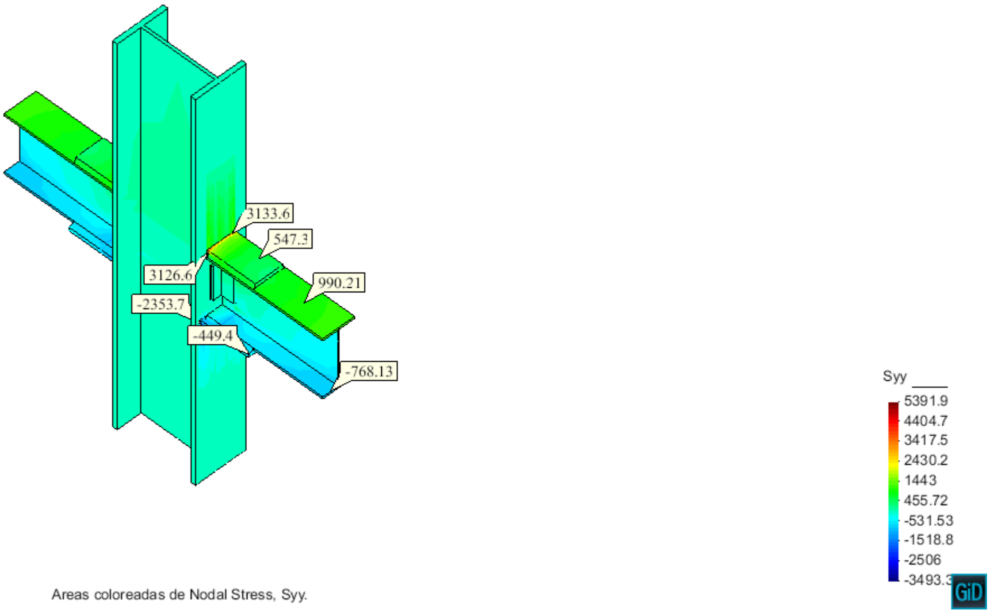


Figure 120: Vertical displacements (Z-axis) in M0-A572 N-QLF model.

Finite Element Analysis results: Model 0, Tension stresses S_{yy}.



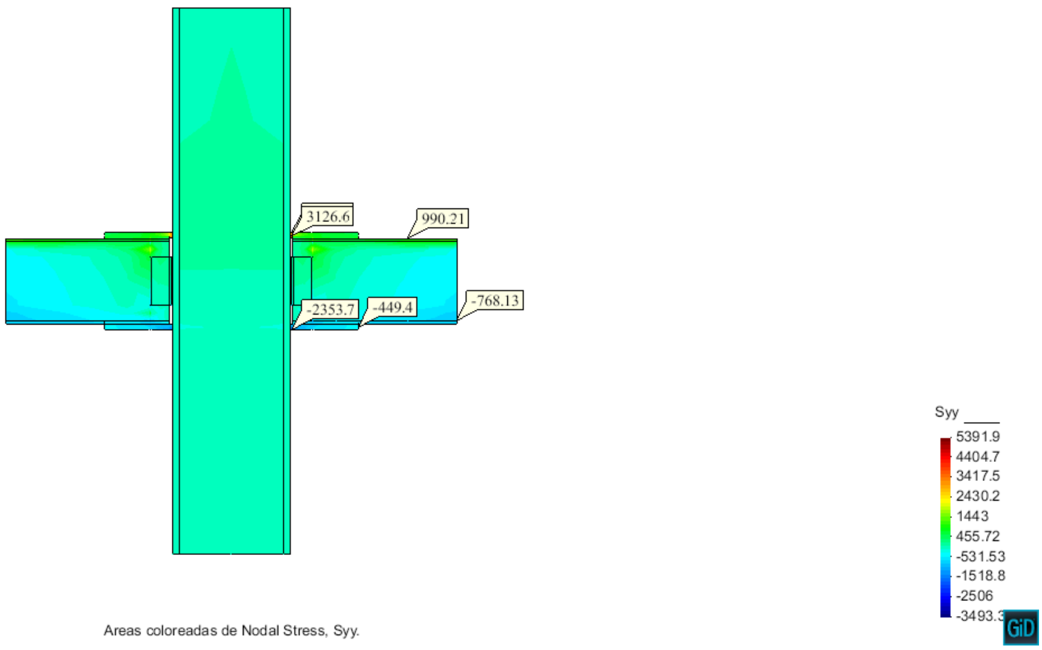


Figure 121: Axial stresses (Y-axis, SYY) in M0-A572 N-QLF model.

Observations:

- Connection have a good performance to bending.

Finite Element Analysis results: Model 0, Shear stresses S_{xy}.

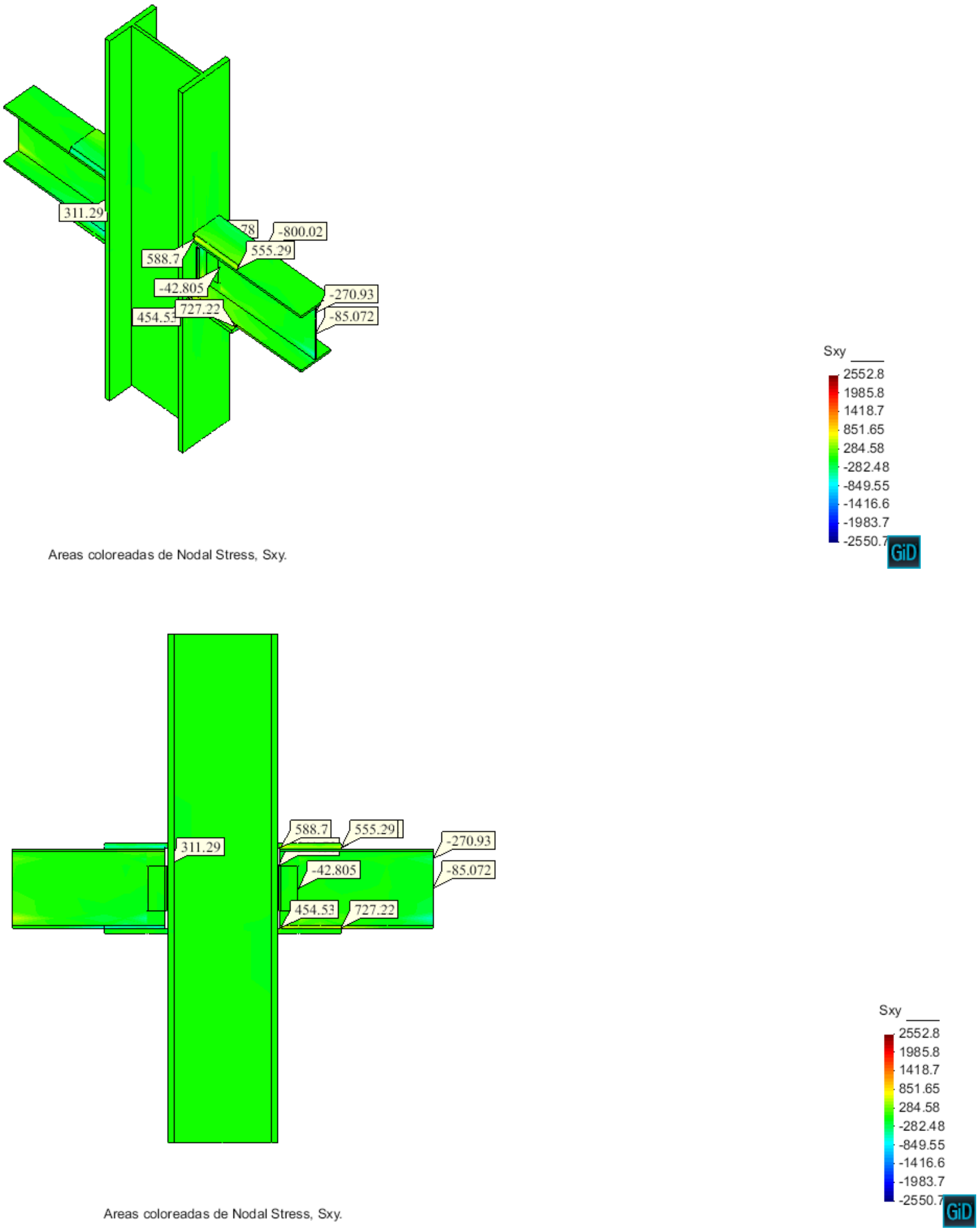
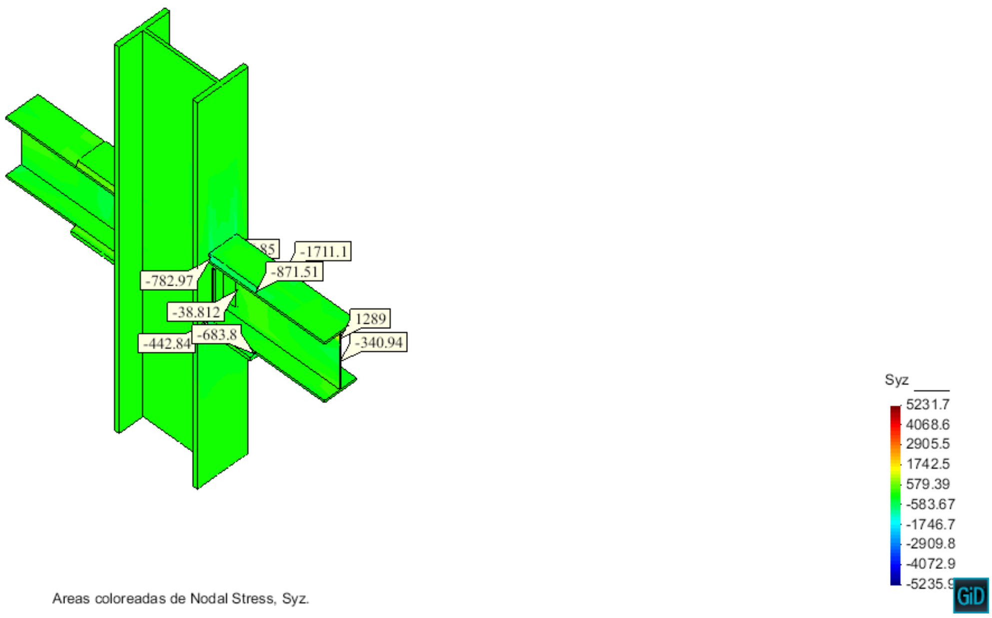


Figure 122: Shear stresses (XY plane, SXY) in M0-A572 N-QLF model.

Observations:

- Shear stresses are acceptable.

Finite Element Analysis results: Model 0, Shear stresses S_{yz}.



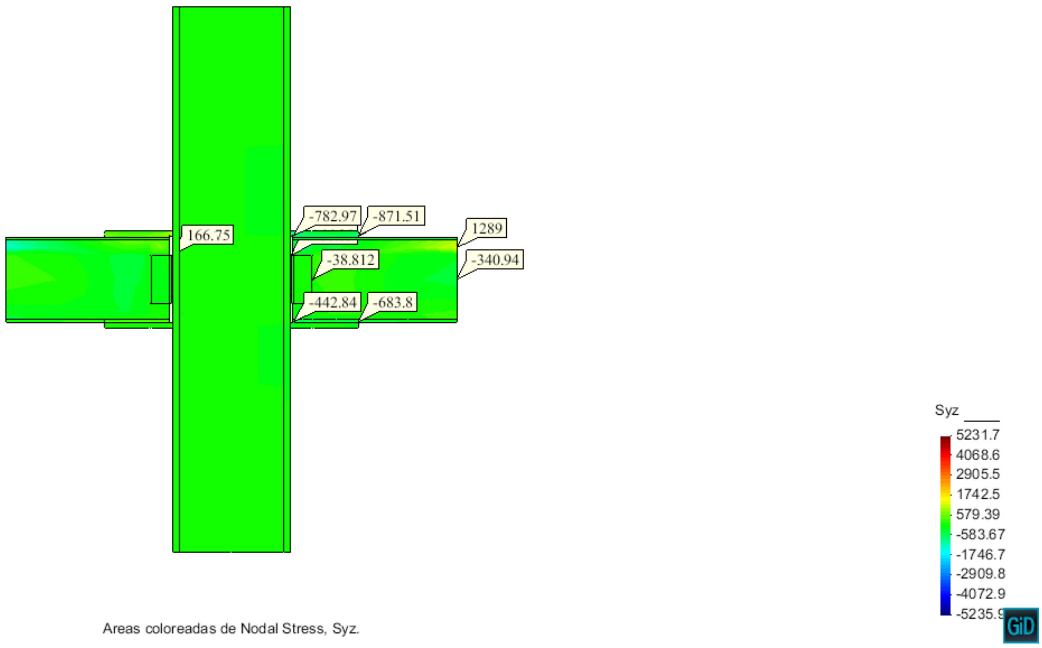
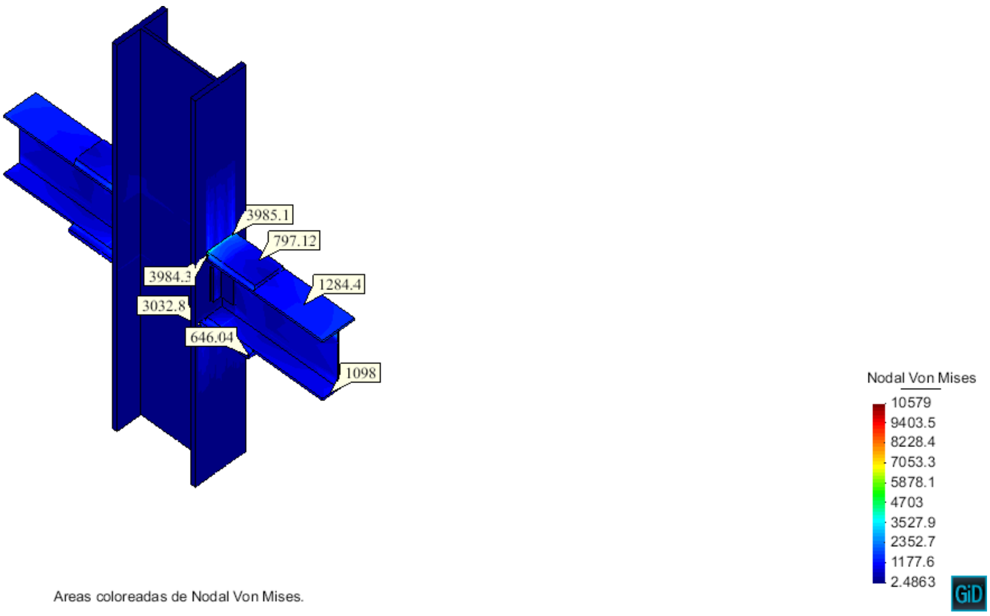


Figure 123: Shear stresses (YZ plane, SYZ) in M0-A572 N-QLF model.

Observations:

- The stress design is overtaken for a 9% for points located at extreme fillets in the small opening between upper rigid plate and the flange of the beam.

Finite Element Analysis results: Model 0, Von Mises stresses VM_s.



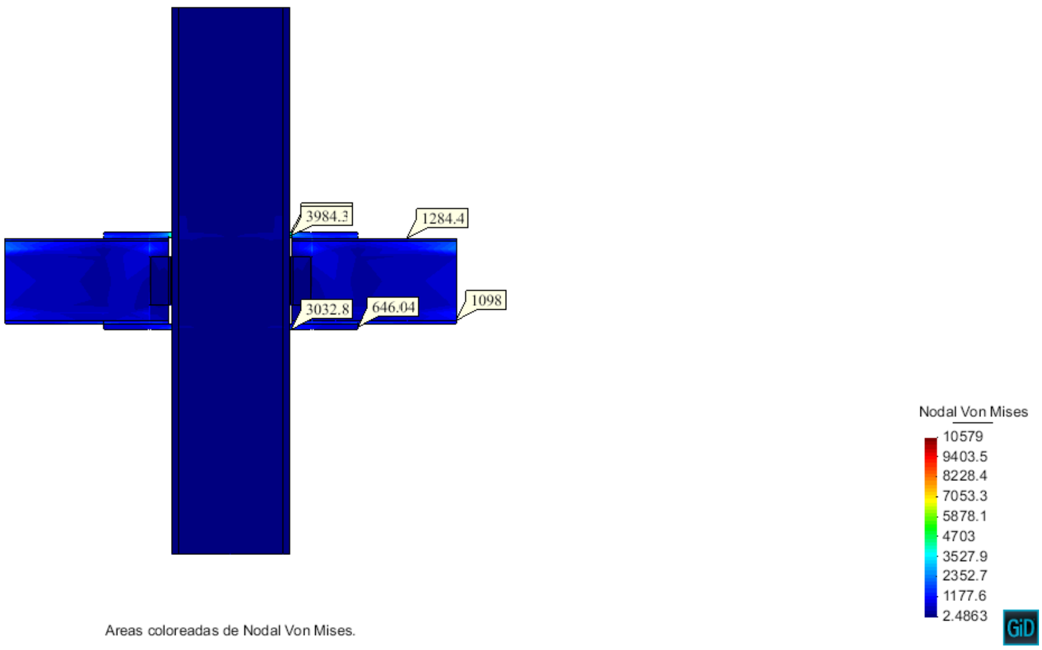


Figure 124: Von Mises stresses in M0-A572 N-QLF model.

Observations:

- There is not pattern formed in the web of the column compared to the correspondent conventional configuration.

7.2.2 Model 1.

Input Data

Name of model: M1-BM1-CL1-A572GR50, (model for offices building)
Beam: (BM_1) W16x45 — Column: (CL_1) W16x100 — Steel Designation: ASTM-A572 Gr. 50
F_y= $3515 kg/cm^2$ ($50 ksi$) — F_u= $4570 kg/cm^2$ ($65 ksi$) — Thickness governant: Column
The flange of beam fits in the flange of column ($0.85 b_{fc} > b_{fb}$).
From beam:

- M_{pf}: $3.62\ e+06\ kg \cdot cm$ flanges plastic moment contribution.
- M_{pw}: $1.11\ e+06\ kg \cdot cm$ web plastic moment contribution.
- M_p: $4.74\ e+06\ kg \cdot cm$ total plastic moment.
- M_y: $4.19\ e+06\ kg \cdot cm$ yield moment.
- $\dfrac{M_{pf}}{M_p} = 76.50\%$ percentage contribution of flanges respect to total plastic moment.
- $\dfrac{M_{pw}}{M_p} = 23.50\%$ percentage contribution of web respect to total plastic moment.

Design forces:

- $V_u = 3.01\ e+04\ kg$ ultimate shear load.
- $V_s = 1.74\ e+04\ kg$ service shear load.
- $M_u = 3.81\ e+06\ kg \cdot cm$ ultimate moment.
- $M_s = 2.26\ e+06\ kg \cdot cm$ service moment.

Rigid Upper Plate

From beam:

Flexural strength.		
$F(kg \cdot cm)$	LRFD (ϕM_n)	ASD ($\frac{M_n}{\Omega}$)
Nominal moment	4.26e+06	2.84e+06

$F_{ut} = F_{uc} = 0.89\ e+05\ kg$ ultimate tension/compression force to design bending plates (according to M_u).
$F_{st} = F_{sc} = 0.55\ e+05\ kg$ service tension/compression force to design bending plates (according to M_s).

Resultant dimensions:

- $b_p = 15.56 cm$ ($6.125 in$) width of plate.
- $t_p = 2.22 cm$ ($0.875 in$) thickness of plate.
- $L_p = 35.00 cm$ ($12.00 in$) length of plate.
- $\psi = 1.13$ shape factor.

Upper plate nominal force.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal yield force	1.09e+05	0.73e+05
Nominal rupture force	1.19e+05	0.79e+05

Fillet Welds for Rigid Plates

Results:

- $EXX/FX = 4900kg/cm^2$ $(70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.80cm$ minimum leg size of weld.
- $tw_{max} = 2.02cm$ maximum leg size of weld.
- $w_p = 1.00cm$ leg size of weld for design of project.
- $Lw_{req} = 56.85cm$ length of weld required.
- $Lw_p = 68.00cm$ length of weld to be adjusted to the project.
- $Lw_{eff} = 68.00cm$ effective length of proportioned weld.
- $R_{nw} = 2078.58kg/cm$ theoretical strength of weld.

Nominal strength of fillet weld.		
$F(kg/cm)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal strength	1558.94	1039.29

Nominal force due to fillet welds.		
$F(kg)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal force	1.06e+05	0.71e+05

Rigid Lower Plate

Results:

- $b_p = 21.59cm$ $(8.50in)$ width of plate.
- $t_p = 1.91cm$ $(0.875in)$ thickness of plate.
- $L_p = 35.00cm$ $(12.00in)$ length of plate.
- $r = 0.55cm$ gyration radius of plate.
- $k = 0.65$ effective length factor.
- $kL_p = 22.75cm$ effective length.
- $\frac{kL_p}{r} = 41.37$ slenderness ratio (adimensional).
- $F_e = 11.76\ e+03\ kg/cm^2$ Euler stress.
- $F_{cr} = 3.10\ e+03\ kg/cm^2$ critical stress.
- $P_n = 1.27\ e+05\ kg$ theoretical compression force resisted.

Compression force resisted by lower plate.		
$F(kg)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal force	1.15e+05	0.76e+05

Shear Design

Beam checking:

- $\dfrac{h}{t_w}$ = 46.67 slenderness ratio of web.
- k_v = 5.34 (a=0, no transverse stiffeners) adimensional value.
- C_{v1} = 1.00 adimensional value.
- $A_w = 35.84cm^2$ area of the web.

Shear strength of beam section.		
$F(kg)$	LRFD (ϕV_n)	ASD ($\frac{V_n}{\Omega}$)
Nominal shear resistance	6.80e+04	4.53e+04

Angles:

- Angle: $L3\ x\ 2\frac{1}{2}\ x\ \frac{3}{8}$
- $b_p = 7.62cm\ (3.00in)$ width of plate.
- $t_p = 0.95cm\ (0.375in)$ thickness of plate.
- $L_p\ or\ h = 25.40cm\ (10.00in)$ length of plate.

Strength of "back-to-back" angles (the pair).		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal yield strength	8.27e+04	5.50e+04
Nominal rupture strength	8.96e+04	5.97e+04

Results for fillet welds of angles:

- $EXX/FX = 4900kg/cm^2\ (70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.50cm$ minimum leg size of weld.
- $tw_{max} = 0.75cm$ maximum leg size of weld.
- $w_p = 0.50cm$ leg size of weld for design of project.
- $Lw_p = 50.80cm$ length of weld proportioned.
- $Lw_{eff} = 50.80cm$ effective length of proportioned weld.

Nominal strength of fillet welds for angles.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal strength	3.17e+04	2.11e+04

Additional quantities.		
Weight	*Length of fillet weld (total)*	*Length of groove weld (total)*
26.23 Kg	101.60 cm of 5.0 mm, and 136.00 cm of 10 mm	37.15 cm

Flanges and Webs with concentrated forces (Chapter J.10)

Flange local bending.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	4.07e+04	2.71e+04
Column	12.37e+04	8.23e+04

From column:

- t_w = $1.49cm$ thickness of web.
- k = $3.53cm$ eccentricity of W shape.
- l_b = $3.91cm$ bearing length (min(t_p,t_{fb})+2w); where:
  - w is the leg size of weld project (w_p).
  - t_p is the thicker thickness of the bending plates.
  - t_{fb} is the thickness of the flange beam.

Web local yielding.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	10.90e+04	7.27e+04

From column:

- t_f = $2.50cm$ thickness of flange.
- h = $43.18cm$ depth of W shape.
- l_b = $3.91cm$ bearing length.
- Q_f = 1.00 adimensional factor.

Web local crippling.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	16.36e+04	10.91e+04

Transverse stiffeners (Chapter G2, Section 3 from ANSI/AISC-360-16):

Feature	**Beam**	**Column**
h	40.89cm	43.18cm
t_w	0.88cm	1.49cm
$\frac{h}{t_w}$	46.67	29.06
$2.46\sqrt{\frac{E}{F_{yw}}}$	59.26	59.26
Requirement?	NO	NO

- F_{yw} = $3515kg/cm^2$ yield stress.
- Transverse stiffeners **not necessary if**:

$$\frac{h}{t_w} < 2.46\sqrt{\frac{E}{F_{yw}}}$$

Stiffening/Continuity Plates

- $C_u = T_u$: 0.00
- A_{req}: $2.68cm^2$
- $\frac{b_{fb}}{3}$: $5.96cm$
- $0.56\sqrt{\frac{E}{F_y}}$: 13.49

For upper plate ($t_{up} = 2.22cm$ $(0.875in)$):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	23.47	34.37	10.56	7.42e+04	4.94e+04
Mid plate	11.73	34.37	5.28	3.71e+04	2.47e+04
Optimal	0.00	34.37	0.00	0.00	0.00

For lower plate ($t_{lp} = 1.91cm$ $(0.75in)$):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	20.12	34.37	10.56	6.36e+04	4.24e+04
Mid plate	10.06	34.37	5.28	3.18e+04	2.12e+04
Optimal	0.00	34.37	0.00	0.00	0.00

Note: strengths (kg) are expressed for just one plate.
For welds (two alternatives, it considers upper and lower edges):

- t_{w1} (weld leg size for alternative 1) $= 0.30cm$ $(\frac{1}{8})''$
- t_{w2} (weld leg size for alternative 2) $= 0.50cm$

Plate	$LRFD(\phi R_{n1})$	$ASD(\frac{R_{n1}}{\Omega})$	$LRFD(\phi R_{n2})$	$ASD(\frac{R_{n2}}{\Omega})$
FP/MP/OP	3.21e+04	2.14e+04	5.36e+04	3.57e+04

Observations

- Strength is governed by fillet welds capacity.
- The forces supported by bending plates and fillet welds have barely a difference estimated by 8%.
- The upper plate works approximately an 80%, and the lower a 10% less than this latter.
- According to calculations, continuity plates can not be implemented.

Finite Element Analysis results: Model 1, Displacements.

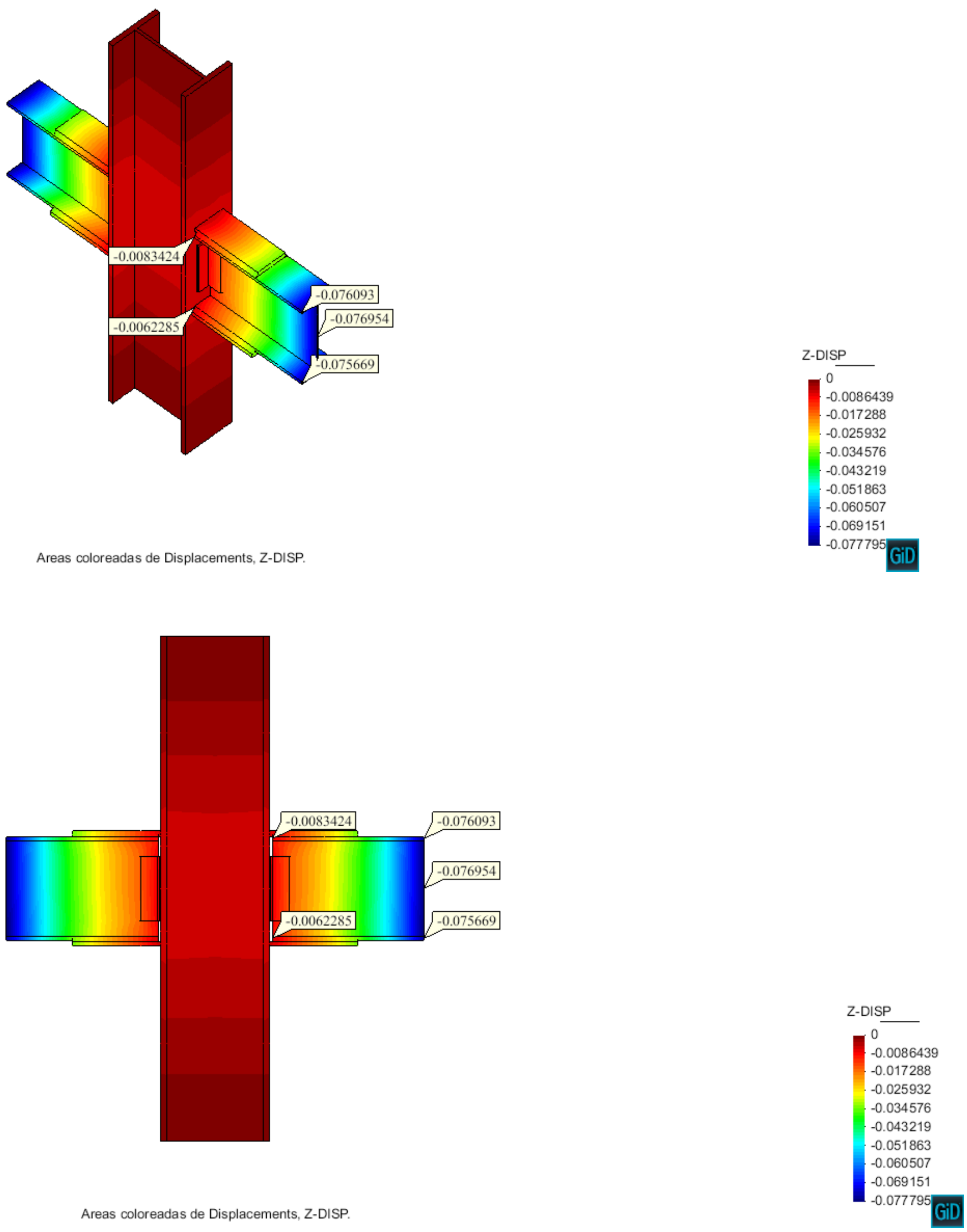
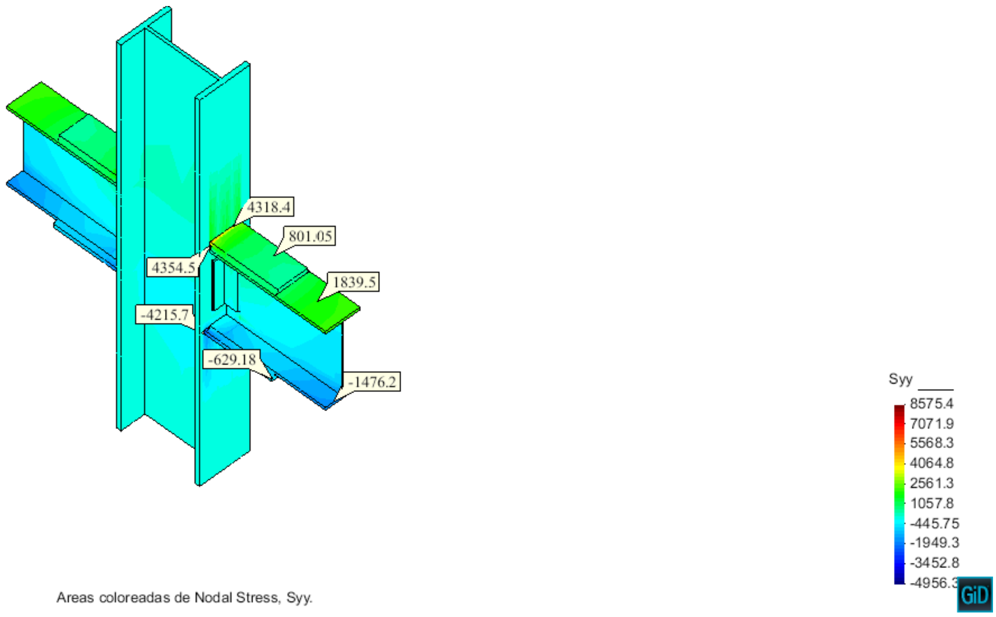


Figure 125: Vertical displacements (Z-axis) in M1-A572 N-QLF model.

Finite Element Analysis results: Model 1, Tension stresses S_{yy}.



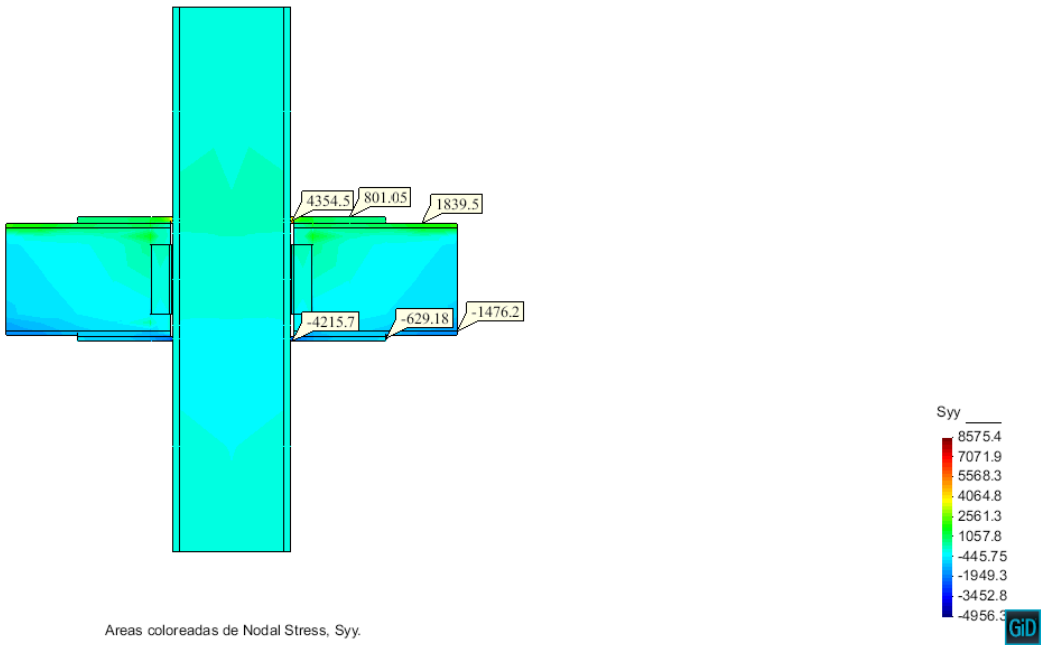
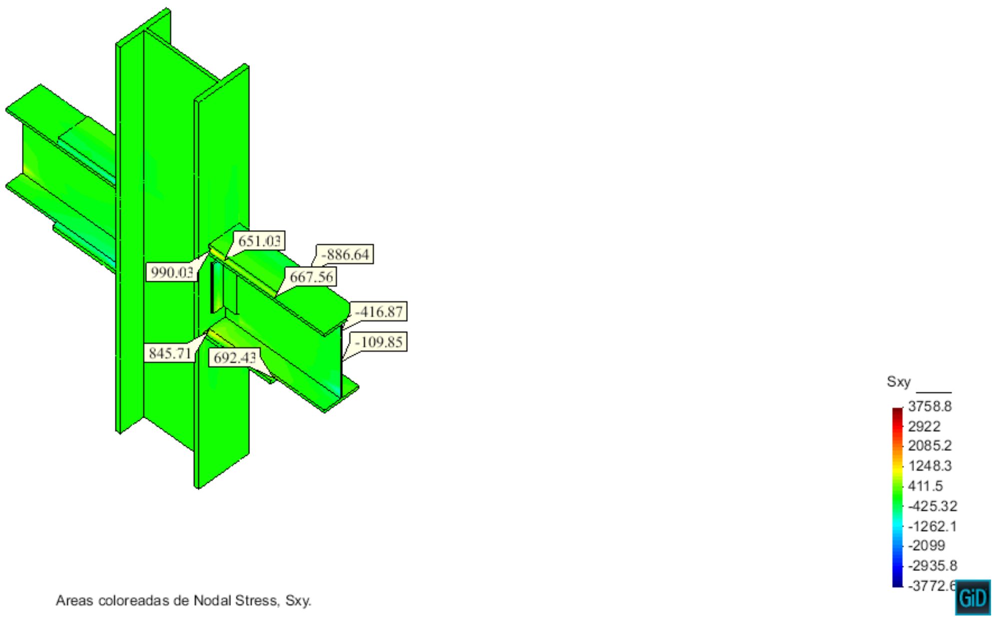


Figure 126: Axial stresses (Y-axis, SYY) in M1-A572 N-QLF model.

Observations:

- The bending plates show stresses around 20% up to limit parameter approximately.
- Connection is not efficient to flexure.

Finite Element Analysis results: Model 1, Shear stresses S_{xy}.



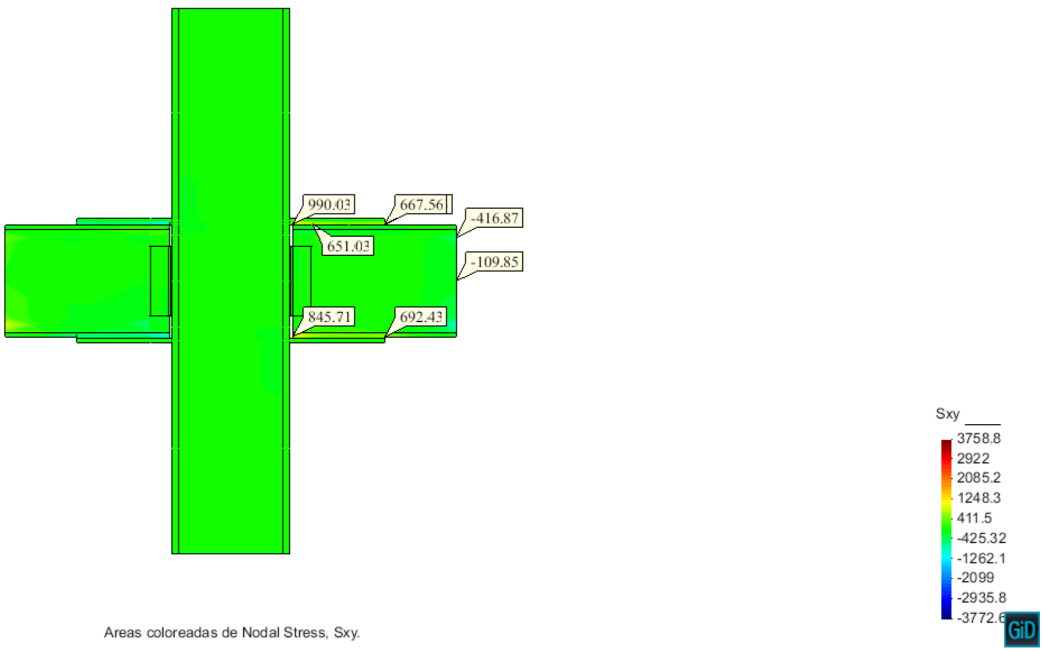
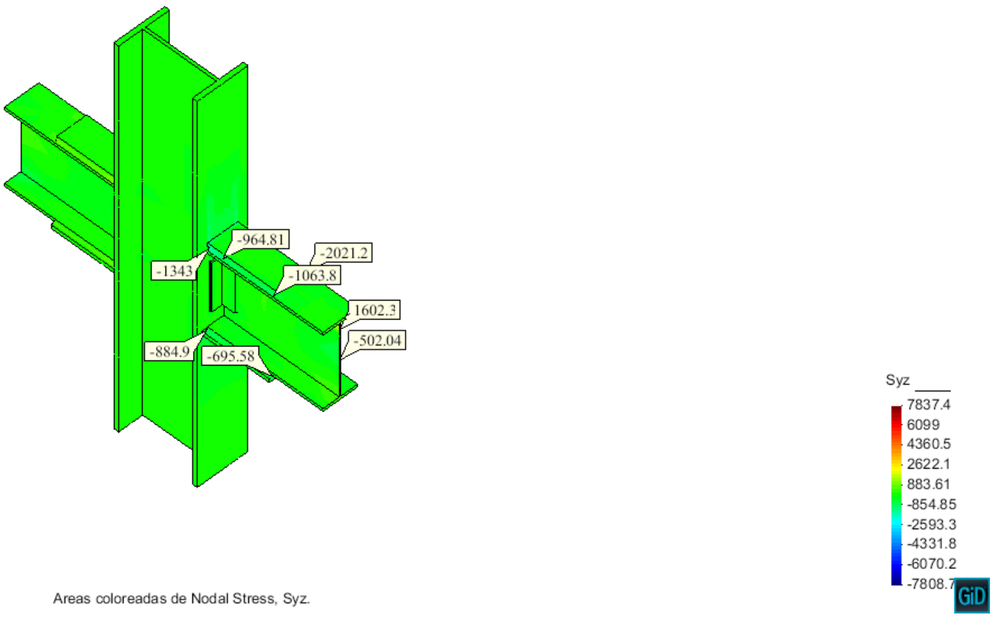


Figure 127: Shear stresses (XY plane, SXY) in M1-A572 N-QLF model.

Observations:

- Values of shear stress obtained are allowable.

Finite Element Analysis results: Model 1, Shear stresses S_{yz}.



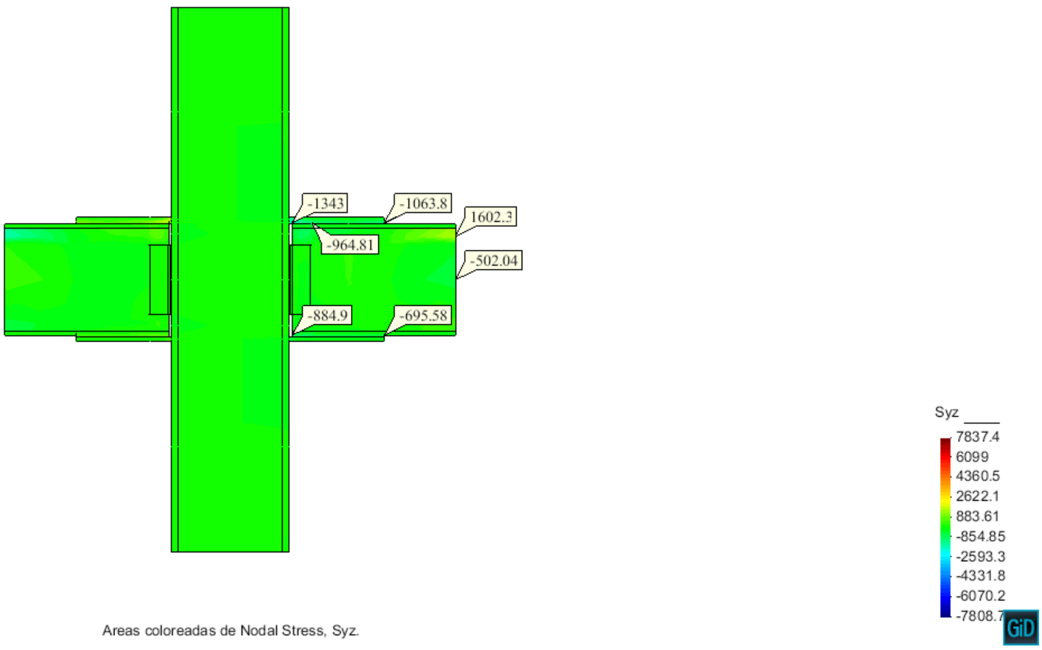


Figure 128: Shear stresses (YZ plane, SYZ) in M1-A572 N-QLF model.

Observations:

- Values of shear stress obtained are allowable, except for extreme edges of upper fillet welds with stresses near to theoretical and overpassing the design limit figure.

Finite Element Analysis results: Model 1, Von Mises stresses VM_s.

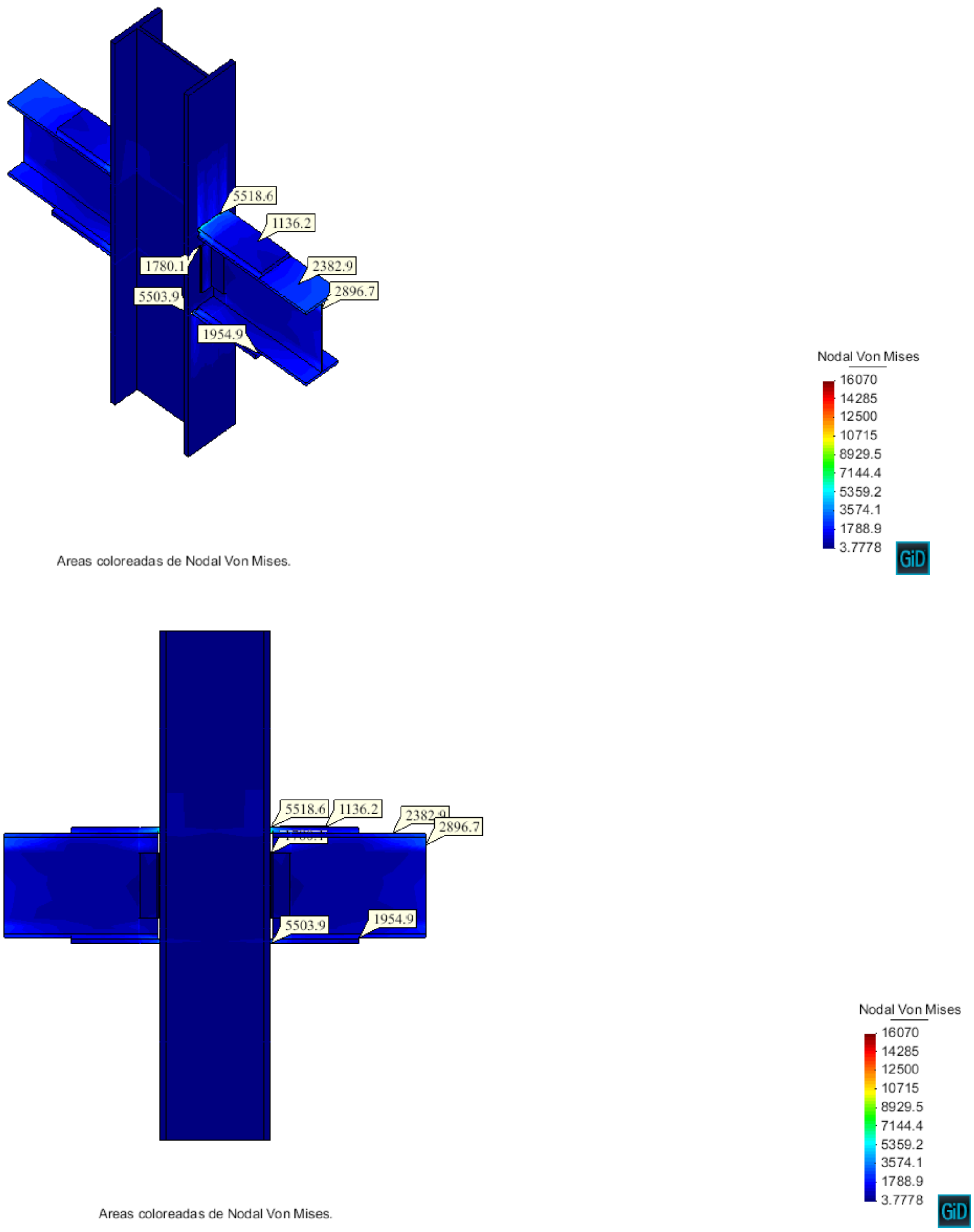


Figure 129: Von Mises stresses in M1-A572 N-QLF model.

Observations:

- Those stresses spread over the plates subjected to couple of forces due to bending are exceeding even the ultimate tensile stress.
- There is an U shape formed in the web of the column that shows certain concentration of stress, and this pattern extends to the flanges.

7.2.3 Model 2.

Input Data

Name of model: M2-BM2-CL2-A572GR50, (model for library building)
Beam: (BM_2) W18x46 — Column: (CL_2) W16x100 — Steel Designation: ASTM-A572 Gr. 50
F_y= $3515 kg/cm^2$ $(50ksi)$ — F_u= $4570kg/cm^2$ $(65ksi)$ — Thickness governant: Column
The flange of beam fits in the flange of column $(0.85 b_{fc} > b_{fb})$.
From beam:

- M_{pf}: $3.76\ e+06\ kg \cdot cm$ flanges plastic moment contribution.
- M_{pw}: $1.48\ e+06\ kg \cdot cm$ web plastic moment contribution.
- M_p: $5.24\ e+06\ kg \cdot cm$ total plastic moment.
- M_y: $4.54\ e+06\ kg \cdot cm$ yield moment.
- $\dfrac{M_{pf}}{M_p} = 71.76\%$ percentage contribution of flanges respect to total plastic moment.
- $\dfrac{M_{pw}}{M_p} = 28.24\%$ percentage contribution of web respect to total plastic moment.

Design forces:

- $V_u = 2.99\ e+04\ kg$ ultimate shear load.
- $V_s = 2.30\ e+04\ kg$ service shear load.
- $M_u = 3.34\ e+06\ kg \cdot cm$ ultimate moment.
- $M_s = 2.57\ e+06\ kg \cdot cm$ service moment.

Rigid Upper Plate

From beam:

Flexural strength.		
$F(kg \cdot cm)$	LRFD (ϕM_n)	ASD $(\frac{M_n}{\Omega})$
Nominal moment	4.71e+06	3.13e+06

$F_{ut} = F_{uc} = 0.85\ e+05\ kg$ ultimate tension/compression force to design bending plates (according to M_u).
$F_{st} = F_{sc} = 0.56\ e+05\ kg$ service tension/compression force to design bending plates (according to M_s).

Resultant dimensions:

- $b_p = 13.02cm$ $(5.125in)$ width of plate.
- $t_p = 2.54cm$ $(1.00in)$ thickness of plate.
- $L_p = 35.00cm$ $(14.00in)$ length of plate.
- $\psi = 1.15$ shape factor.

Upper plate nominal force.		
$F(kg)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal yield force	1.05e+05	0.70e+05
Nominal rupture force	1.13e+05	0.76e+05

Fillet Welds for Rigid Plates

Results:

- $EXX/FX = 4900 kg/cm^2$ $(70 ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.80 cm$ minimum leg size of weld.
- $tw_{max} = 2.34 cm$ maximum leg size of weld.
- $w_p = 1.00 cm$ leg size of weld for design of project.
- $Lw_{req} = 54.81 cm$ length of weld required.
- $Lw_p = 68.00 cm$ length of weld to be adjusted to the project.
- $Lw_{eff} = 68.00 cm$ effective length of proportioned weld.
- $R_{nw} = 2078.58 kg/cm$ theoretical strength of weld.

Nominal strength of fillet weld.		
$F(kg/cm)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal strength	1558.94	1039.29

Nominal force due to fillet welds.		
$F(kg)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal force	1.06e+05	0.71e+05

Rigid Lower Plate

Results:

- $b_p = 17.78 cm$ $(7.00 in)$ width of plate.
- $t_p = 1.91 cm$ $(0.75 in)$ thickness of plate.
- $L_p = 35.00 cm$ $(14.00 in)$ length of plate.
- $r = 0.55 cm$ gyration radius of plate.
- $k = 0.65$ effective length factor.
- $kL_p = 22.75 cm$ effective length.
- $\frac{kL_p}{r} = 41.37$ slenderness ratio (adimensional).
- $F_e = 11.76\ e+03\ kg/cm^2$ Euler stress.
- $F_{cr} = 3.10\ e+03\ kg/cm^2$ critical stress.
- $P_n = 1.05\ e+05\ kg$ theoretical compression force resisted.

Compression force resisted by lower plate.		
$F(kg)$	LRFD (ϕR_n)	ASD $(\frac{R_n}{\Omega})$
Nominal force	0.95e+05	0.63e+05

Shear Design

Beam checking:

- $\frac{h}{t_w}$ = 50.28 slenderness ratio of web.
- k_v = 5.34 (a=0, no transverse stiffeners) adimensional value.
- C_{v1} = 1.00 adimensional value.
- $A_w = 42.04cm^2$ area of the web.

Shear strength of beam section.		
$F(kg)$	LRFD (ϕV_n)	ASD ($\frac{V_n}{\Omega}$)
Nominal shear resistance	7.98e+04	5.31e+04

Angles:

- Angle: $L3\ x\ 2\ x\ \frac{3}{8}$
- $b_p = 5.08cm$ $(2.00in)$ width of plate.
- $t_p = 0.95cm$ $(0.375in)$ thickness of plate.
- $L_p\ or\ h = 25.40cm$ $(10.00in)$ length of plate.

Strength of "back-to-back" angles (the pair).		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal yield strength	9.18e+04	6.11e+04
Nominal rupture strength	9.95e+04	6.63e+04

Results for fillet welds of angles:

- $EXX/FX = 4900kg/cm^2$ $(70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.50cm$ minimum leg size of weld.
- $tw_{max} = 0.75cm$ maximum leg size of weld.
- $w_p = 0.50cm$ leg size of weld for design of project.
- $Lw_p = 50.80cm$ length of weld proportioned.
- $Lw_{eff} = 50.80cm$ effective length of proportioned weld.

Nominal strength of fillet welds for angles.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal strength	3.17e+04	2.11e+04

Additional quantities.		
Weight	*Length of fillet weld (total)*	*Length of groove weld (total)*
22.89 Kg	101.60 cm of 5.0 mm, and 136.00 cm of 10 mm	30.80 cm

Flanges and Webs with concentrated forces (Chapter J.10)

Flange local bending.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	4.67e+04	3.11e+04
Column	12.37e+04	8.23e+04

From column:

- t_w = 1.49cm thickness of web.
- k = 3.53cm eccentricity of W shape.
- l_b = 3.91cm bearing length (min(t_p,t_{fb})+2w); where:
  - w is the leg size of weld project (w_p).
  - t_p is the thicker thickness of the bending plates.
  - t_{fb} is the thickness of the flange beam.

Web local yielding.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	10.74e+04	7.16e+04

From column:

- t_f = 2.50cm thickness of flange.
- h = 43.18cm depth of W shape.
- l_b = 3.91cm bearing length.
- Q_f = 1.00 adimensional factor.

Web local crippling.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	16.36e+04	10.91e+04

Transverse stiffeners (Chapter G2, Section 3 from ANSI/AISC-360-16):

Feature	**Beam**	**Column**
h	45.97cm	43.18cm
t_w	0.91cm	1.49cm
$\frac{h}{t_w}$	50.28	29.06
$2.46\sqrt{\frac{E}{F_{yw}}}$	59.26	59.26
Requirement?	NO	NO

- F_{yw} = 3515kg/cm^2 yield stress.
- Transverse stiffeners **not necessary if**:

$$\frac{h}{t_w} < 2.46\sqrt{\frac{E}{F_{yw}}}$$

Stiffening/Continuity Plates

- $C_u = T_u$: 7ton
- A_{req}: $2.07cm^2$
- $\frac{b_{fb}}{3}$: $5.96cm$
- $0.56\sqrt{\frac{E}{F_y}}$: 13.49

For upper plate ($t_{up} = 2.54cm$ $(1.00in)$):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	26.82	34.37	10.56	8.48e+04	5.65e+04
Mid plate	13.41	34.37	5.28	4.24e+04	2.83e+04
Optimal	0.00	34.37	0.00	0.00	0.00

For lower plate ($t_{lp} = 1.91cm$ $(0.75in)$):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	20.12	34.37	10.56	6.36e+04	4.24e+04
Mid plate	10.06	34.37	5.28	3.18e+04	2.12e+04
Optimal	0.00	34.37	0.00	0.00	0.00

Note: strengths (kg) are expressed for just one plate.
For welds (two alternatives, it considers upper and lower edges):

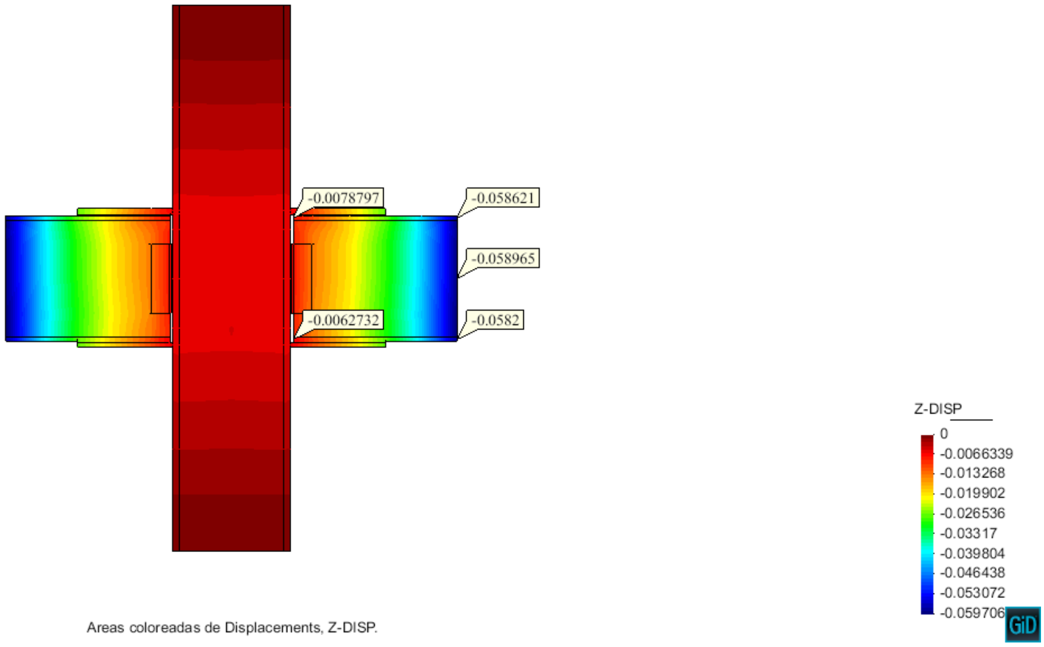
- t_{w1} (weld leg size for alternative 1) $= 0.30cm$ $(\frac{1}{8})''$
- t_{w2} (weld leg size for alternative 2) $= 0.50cm$

Plate	$LRFD(\phi R_{n1})$	$ASD(\frac{R_{n1}}{\Omega})$	$LRFD(\phi R_{n2})$	$ASD(\frac{R_{n2}}{\Omega})$
FP/MP/OP	3.21e+04	2.14e+04	5.36e+04	3.57e+04

Observations

- Both plates reach the same effectiveness (81%) respect to total yield resistance.
- Bending plates are estimated to fail before fillet welds that subject them.
- Fillet welds for shear design almost exceed the 100% of their capacity with barely a 5% less than strength needed.

Finite Element Analysis results: Model 2, Displacements.



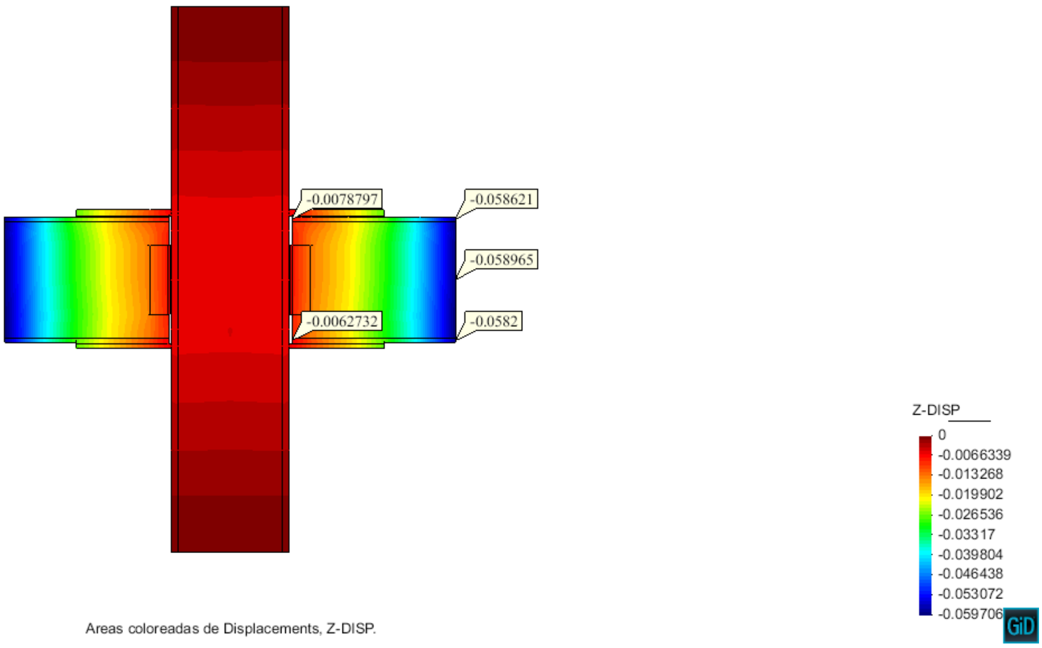
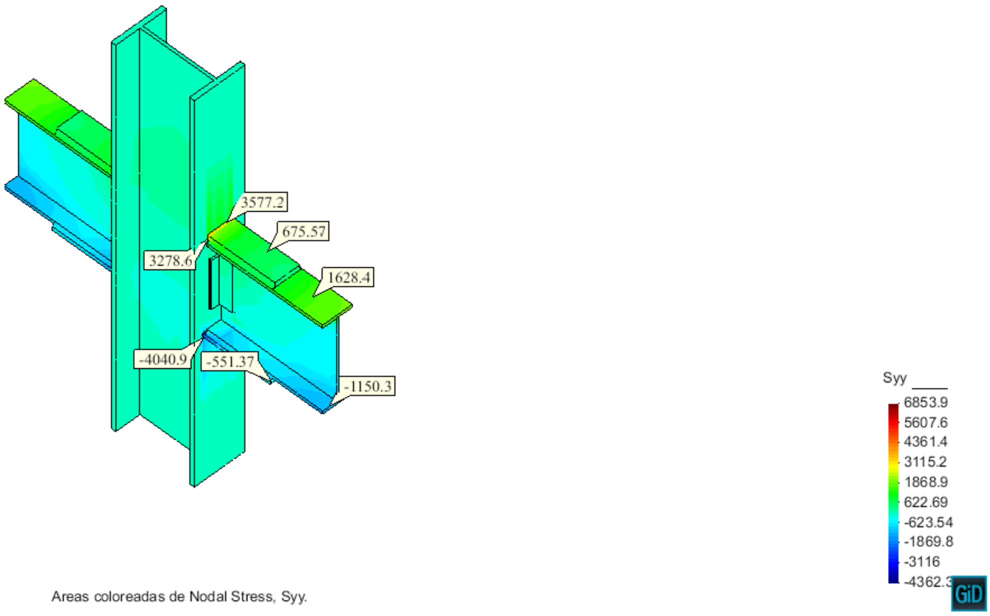


Figure 130: Vertical displacements (Z-axis) in M2-A572 N-QLF model.

Finite Element Analysis results: Model 2, Tension stresses S_{yy}.



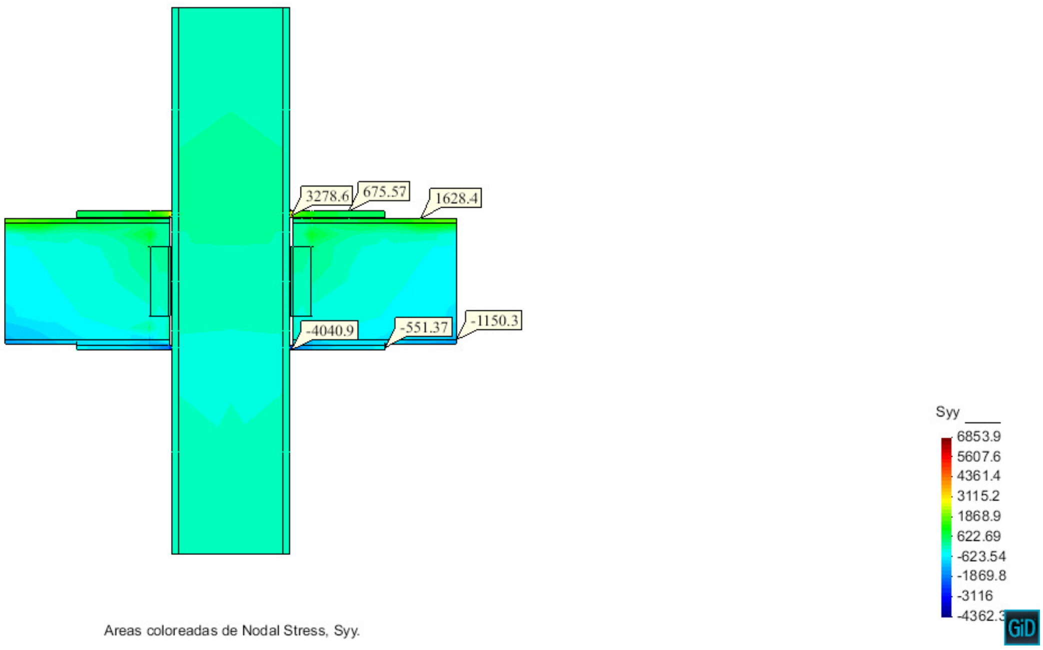
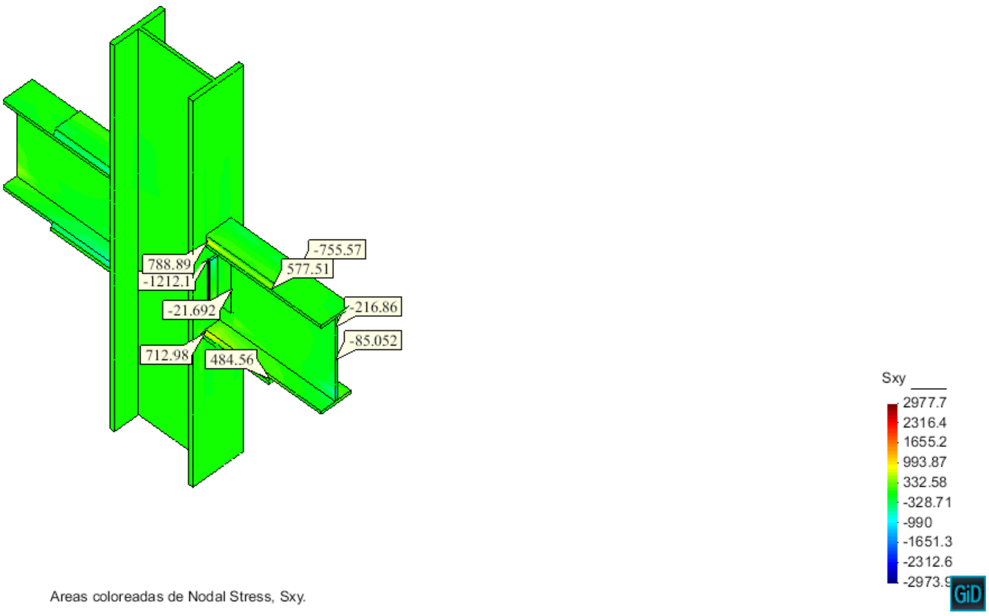


Figure 131: Axial stresses (Y-axis, SYY) in M2-A572 N-QLF model.

Finite Element Analysis results: Model 2, Shear stresses S_{xy}.



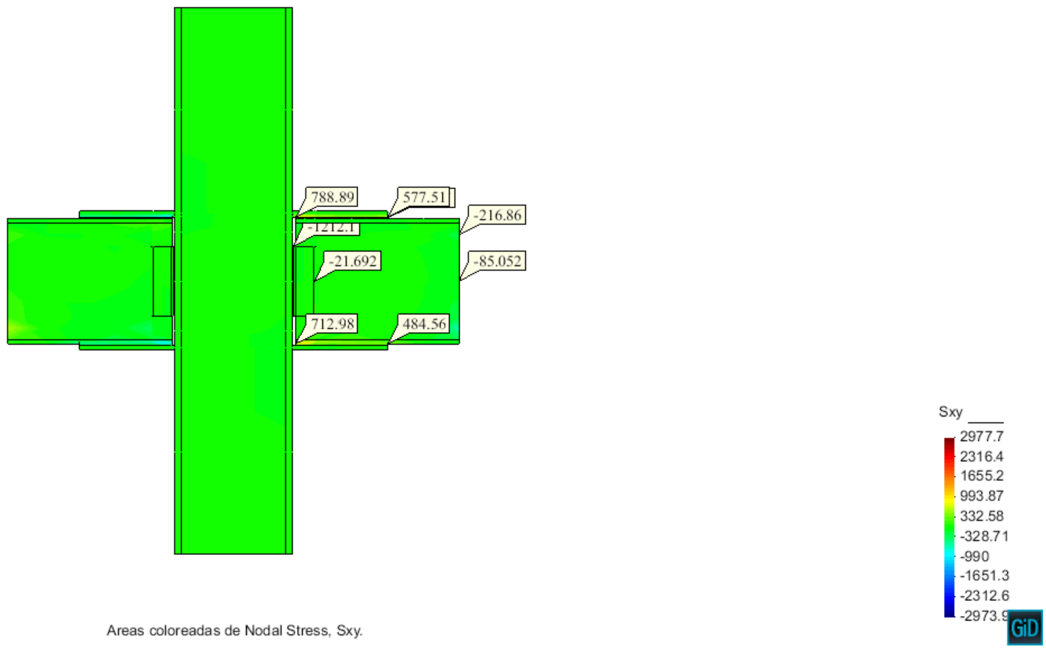


Figure 132: Shear stresses (XY plane, SXY) in M2-A572 N-QLF model.

Finite Element Analysis results: Model 2, Shear stresses S_{yz}.

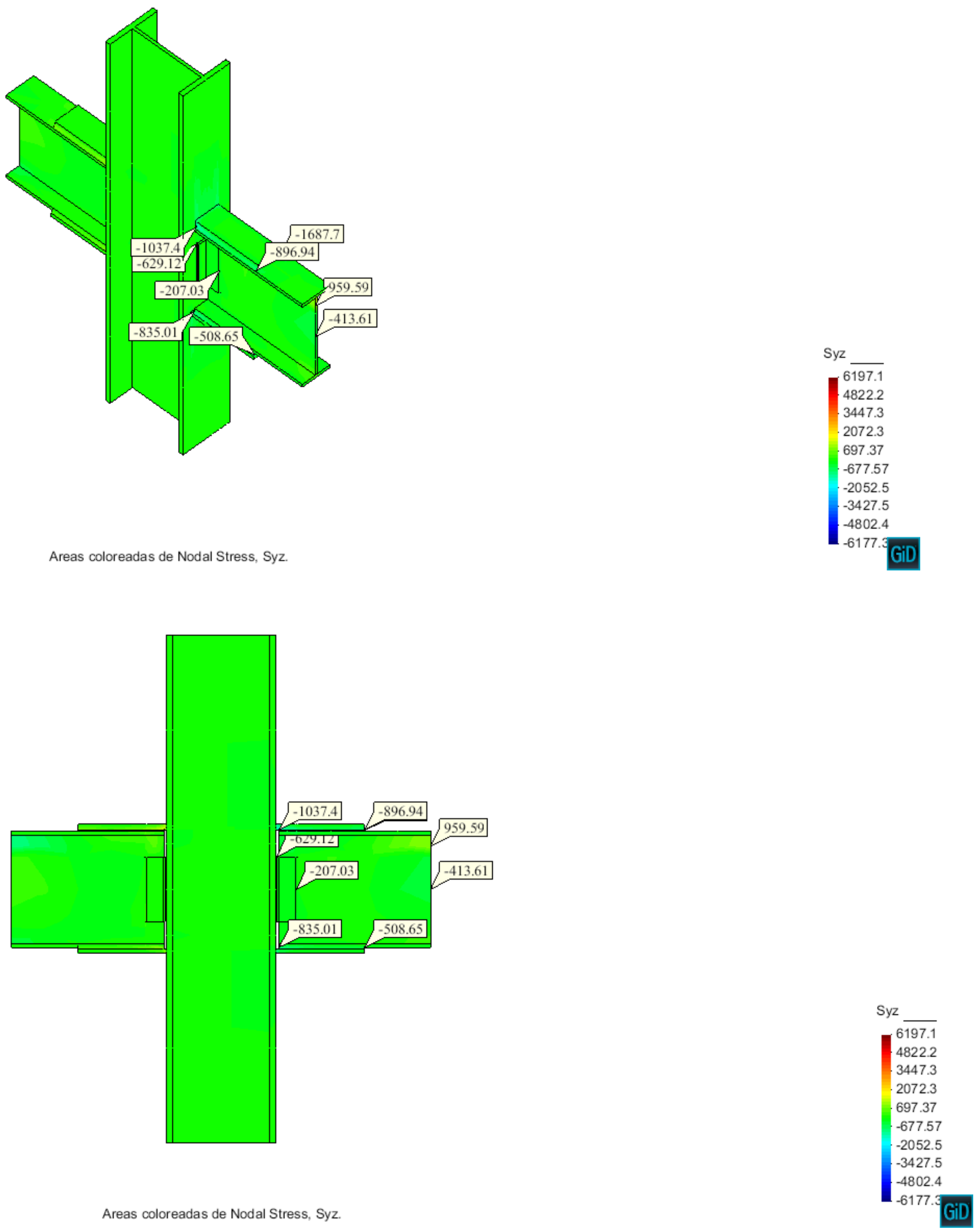
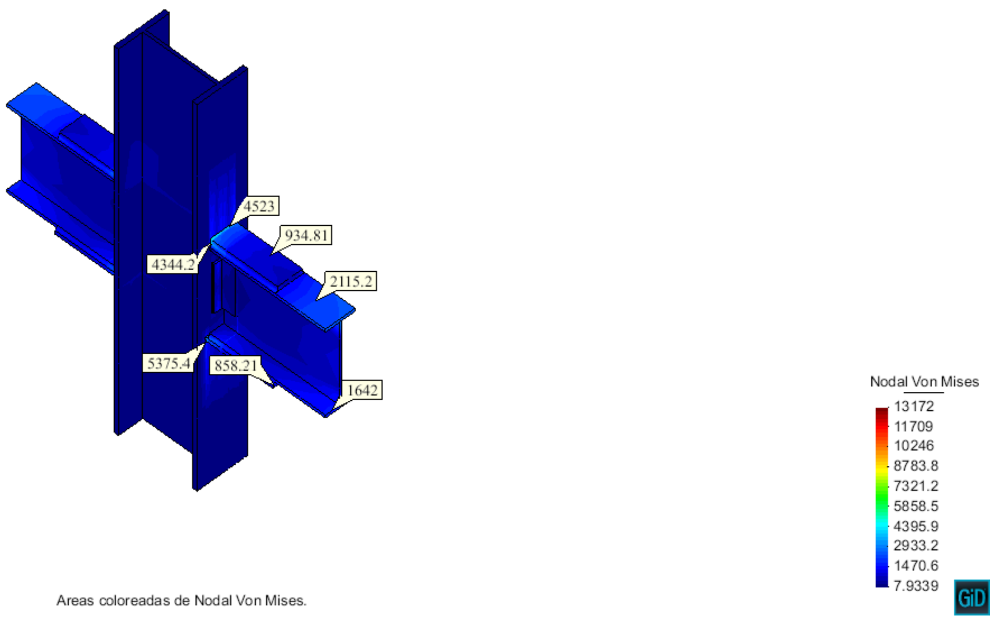


Figure 133: Shear stresses (YZ plane, SYZ) in M2-A572 N-QLF model.

Finite Element Analysis results: Model 2, Von Mises stresses VM_s.



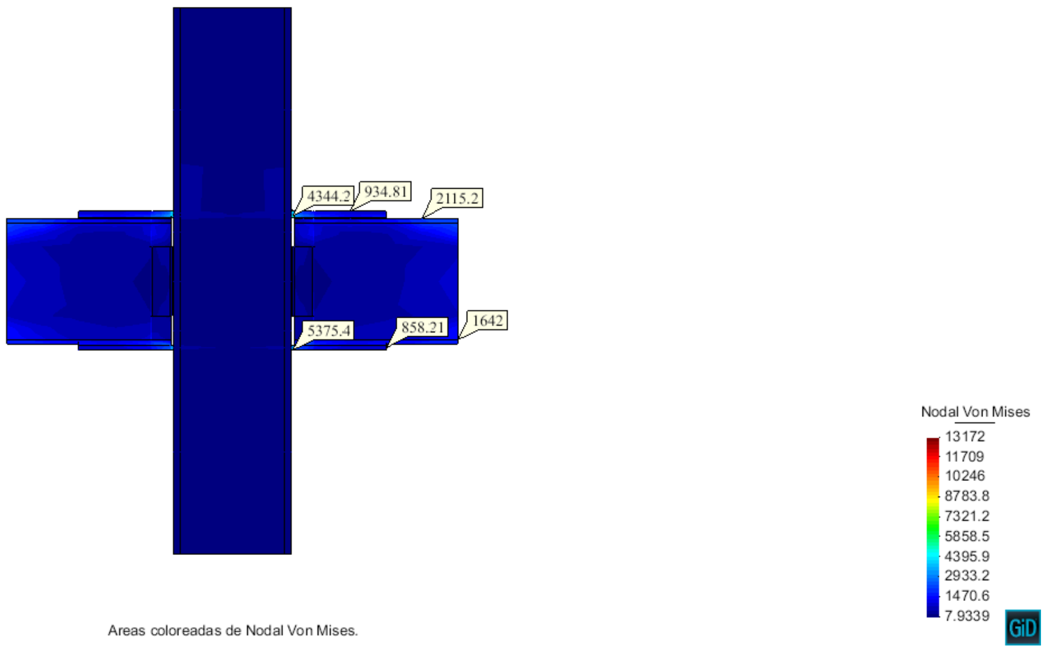


Figure 134: Von Mises stresses in M2-A572 N-QLF model.

7.2.4 Model 3.

Input Data

Name of model: M3-BM3-CL3-A572GR50, (arbitrary model)
Beam: (BM_3) W24x55 — Column: (CL_3) W16x100 — Steel Designation: ASTM-A572 Gr. 50
F_y= $3515kg/cm^2$ $(50ksi)$ — F_u= $4570kg/cm^2$ $(65ksi)$ — Thickness governant: Column
The flange of beam fits in the flange of column $(0.85b_{fc} > b_{fb})$.
From beam:

- M_{pf}: $4.76\ e+06\ kg\cdot cm$ flanges plastic moment contribution.
- M_{pw}: $2.90\ e+06\ kg\cdot cm$ web plastic moment contribution.
- M_p: $7.66\ e+06\ kg\cdot cm$ total plastic moment.
- M_y: $6.57\ e+06\ kg\cdot cm$ yield moment.
- $\dfrac{M_{pf}}{M_p} = 62.12\%$ percentage contribution of flanges respect to total plastic moment.
- $\dfrac{M_{pw}}{M_p} = 37.88\%$ percentage contribution of web respect to total plastic moment.

Design forces:

- $V_u = 3.77\ e+04\ kg$ ultimate shear load.
- $V_s = 2.69\ e+04\ kg$ service shear load.
- $M_u = 5.25\ e+06\ kg\cdot cm$ ultimate moment.
- $M_s = 3.75\ e+06\ kg\cdot cm$ service moment.

Rigid Upper Plate

From beam:

Flexural strength.		
$F(kg\cdot cm)$	LRFD (ϕM_n)	ASD $(\dfrac{M_n}{\Omega})$
Nominal moment	6.90e+06	4.59e+06

$F_{ut} = F_{uc} = 0.96\ e+05\ kg$ ultimate tension/compression force to design bending plates (according to M_u).
$F_{st} = F_{sc} = 0.63\ e+05\ kg$ service tension/compression force to design bending plates (according to M_s).

Resultant dimensions:

- $b_p = 15.56cm\ (6.125in)$ width of plate.
- $t_p = 2.54cm\ (1.00in)$ thickness of plate.
- $L_p = 35.00cm\ (14.00in)$ length of plate.
- $\psi = 1.18$ shape factor.

Upper plate nominal force.		
$F(kg)$	LRFD (ϕR_n)	ASD $(\dfrac{R_n}{\Omega})$
Nominal yield force	1.25e+05	0.83e+05
Nominal rupture force	1.35e+05	0.90e+05

Fillet Welds for Rigid Plates

Results:

- $EXX/FX = 4900 kg/cm^2$ $(70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.80cm$ minimum leg size of weld.
- $tw_{max} = 2.34cm$ maximum leg size of weld.
- $w_p = 1.00cm$ leg size of weld for design of project.
- $Lw_{req} = 61.50cm$ length of weld required.
- $Lw_p = 68.00cm$ length of weld to be adjusted to the project.
- $Lw_{eff} = 68.00cm$ effective length of proportioned weld.
- $R_{nw} = 2078.58kg/cm$ theoretical strength of weld.

Nominal strength of fillet weld.		
$F(kg/cm)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal strength	1558.94	1039.29

Nominal force due to fillet welds.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal force	1.06e+05	0.71e+05

Rigid Lower Plate

Results:

- $b_p = 21.59cm$ $(8.50in)$ width of plate.
- $t_p = 1.91cm$ $(0.75in)$ thickness of plate.
- $L_p = 35.00cm$ $(14.00in)$ length of plate.
- $r = 0.55cm$ gyration radius of plate.
- $k = 0.65$ effective length factor.
- $kL_p = 22.75cm$ effective length.
- $\frac{kL_p}{r} = 41.37$ slenderness ratio (adimensional).
- $F_e = 11.76\ e+03\ kg/cm^2$ Euler stress.
- $F_{cr} = 3.10\ e+03\ kg/cm^2$ critical stress.
- $P_n = 1.28\ e+05\ kg$ theoretical compression force resisted.

Compression force resisted by lower plate.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal force	1.15e+05	0.76e+05

Shear Design

Beam checking:

- $\dfrac{h}{t_w} = 59.75$ slenderness ratio of web.
- $k_v = 5.34$ (a=0, no transverse stiffeners) adimensional value.
- $C_{v1} = 1.00$ adimensional value.
- $A_w = 60.14cm^2$ area of the web.

Shear strength of beam section.		
$F(kg)$	LRFD (ϕV_n)	ASD ($\frac{V_n}{\Omega}$)
Nominal shear resistance	11.42e+04	7.59e+04

Angles:

- Angle: $L3\ x\ 3\ x\ \frac{3}{8}$
- $b_p = 7.62cm\ (3.00in)$ width of plate.
- $t_p = 0.95cm\ (0.375in)$ thickness of plate.
- $L_p\ or\ h = 35.56cm\ (14.00in)$ length of plate.

Strength of "back-to-back" angles (the pair).		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal yield strength	12.86e+04	8.55e+04
Nominal rupture strength	13.93e+04	9.29e+04

Results for fillet welds of angles:

- $EXX/FX = 4900kg/cm^2\ (70ksi)$ strength of electrode/flux employed.
- $tw_{min} = 0.50cm$ minimum leg size of weld.
- $tw_{max} = 0.75cm$ maximum leg size of weld.
- $w_p = 0.50cm$ leg size of weld for design of project.
- $Lw_p = 71.12cm$ length of weld proportioned.
- $Lw_{eff} = 71.12cm$ effective length of proportioned weld.

Nominal strength of fillet welds for angles.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Nominal strength	5.54e+04	3.69e+04

Additional quantities.		
Weight	*Length of fillet weld (total)*	*Length of groove weld (total)*
29.07 Kg	142.24 cm of 5.0 mm, and 136.00 cm of 10 mm	37.15 cm

Flanges and Webs with concentrated forces (Chapter J.10)

Flange local bending.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	3.25e+04	2.16e+04
Column	12.37e+04	8.23e+04

From column:

- $t_w = 1.49cm$ thickness of web.
- $k = 3.53cm$ eccentricity of W shape.
- $l_b = 3.91cm$ bearing length (min(t_p,t_{fb})+2w); where:
  - w is the leg size of weld project (w_p).
  - t_p is the thicker thickness of the bending plates.
  - t_{fb} is the thickness of the flange beam.

Web local yielding.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	10.74e+04	7.16e+04

From column:

- $t_f = 2.50cm$ thickness of flange.
- $h = 43.18cm$ depth of W shape.
- $l_b = 3.91cm$ bearing length.
- $Q_f = 1.00$ adimensional factor.

Web local crippling.		
$F(kg)$	LRFD (ϕR_n)	ASD ($\frac{R_n}{\Omega}$)
Beam	—	—
Column	16.64e+04	11.09e+04

Transverse stiffeners (Chapter G2, Section 3 from ANSI/AISC-360-16):

Feature	**Beam**	**Column**
h	59.94cm	43.18cm
t_w	0.91cm	1.49cm
$\frac{h}{t_w}$	59.75	29.06
$2.46\sqrt{\frac{E}{F_{yw}}}$	59.26	59.26
Requirement?	YES	NO

- $F_{yw} = 3515kg/cm^2$ yield stress.
- Transverse stiffeners **not necessary if**:

$$\frac{h}{t_w} < 2.46\sqrt{\frac{E}{F_{yw}}}$$

Stiffening/Continuity Plates

- $C_u = T_u$: 21.00ton
- A_{req}: $6.47cm^2$
- $\frac{b_{fb}}{3}$: $5.94cm$
- $0.56\sqrt{\frac{E}{F_y}}$: 13.49

For upper plate ($t_{up} = 2.54cm$ ($1.00in$)):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	26.82	34.37	10.56	8.48e+04	5.65e+04
Mid plate	13.41	34.37	5.28	4.24e+04	2.83e+04
Optimal	0.00	34.37	0.00	0.00	0.00

For lower plate ($t_{lp} = 1.91cm$ ($0.75in$)):

Plate	$A(cm^2)$	$L_s(cm)$	$b_s(cm)$	$LRFD(\phi R_n)$	$ASD(\frac{R_n}{\Omega})$
Full plate	20.12	34.37	10.56	6.36e+04	4.24e+04
Mid plate	10.06	34.37	5.28	3.18e+04	2.12e+04
Optimal	0.00	34.37	0.00	0.00	0.00

Note: strengths (kg) are expressed for just one plate.
For welds (two alternatives, it considers upper and lower edges):

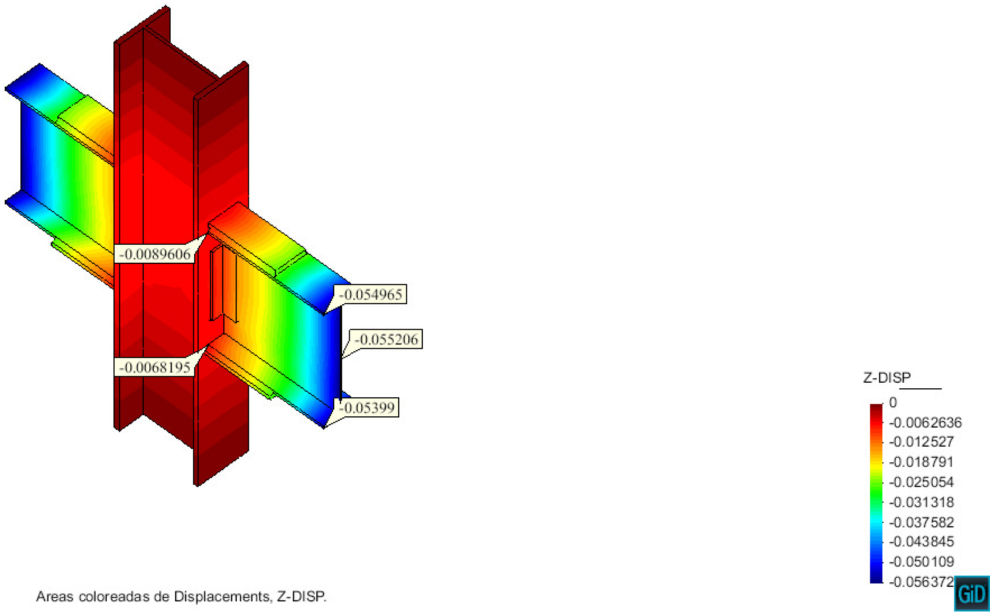
- t_{w1} (weld leg size for alternative 1) = $0.30cm$ $(\frac{1}{8})''$
- t_{w2} (weld leg size for alternative 2) = $0.50cm$

Plate	$LRFD(\phi R_{n1})$	$ASD(\frac{R_{n1}}{\Omega})$	$LRFD(\phi R_{n2})$	$ASD(\frac{R_{n2}}{\Omega})$
FP/MP/OP	3.21e+04	2.14e+04	5.36e+04	3.57e+04

Observations

- Flexural plates reach a similar effectiveness of 75% for demanded capacity.
- Fillet welds rule the connection performance.
- A pair of continuity plates is required.

Finite Element Analysis results: Model 3, Displacements.



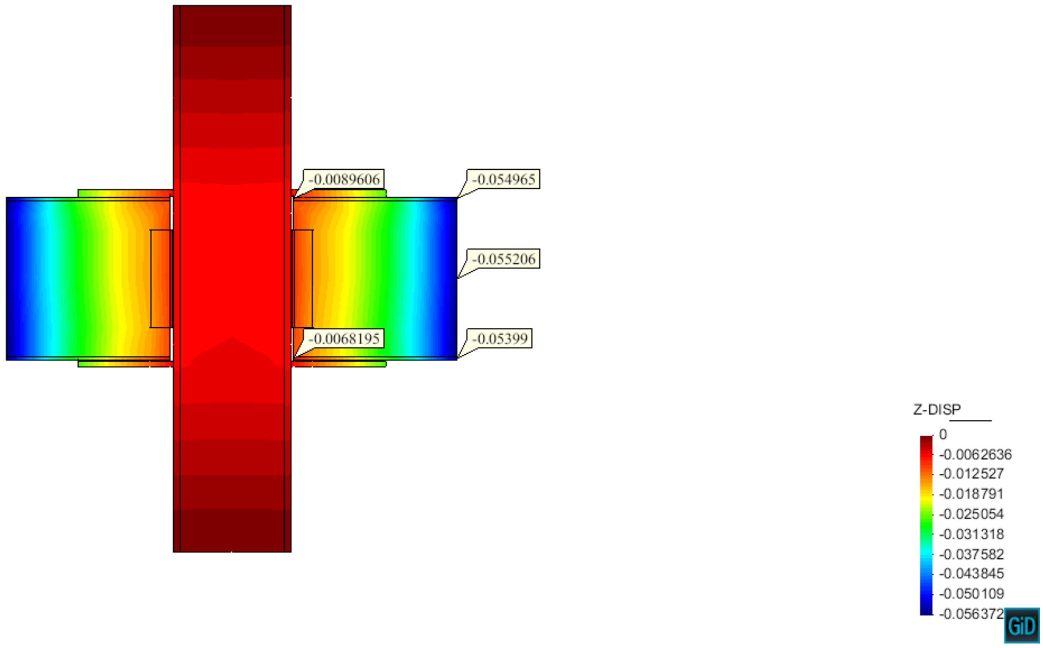
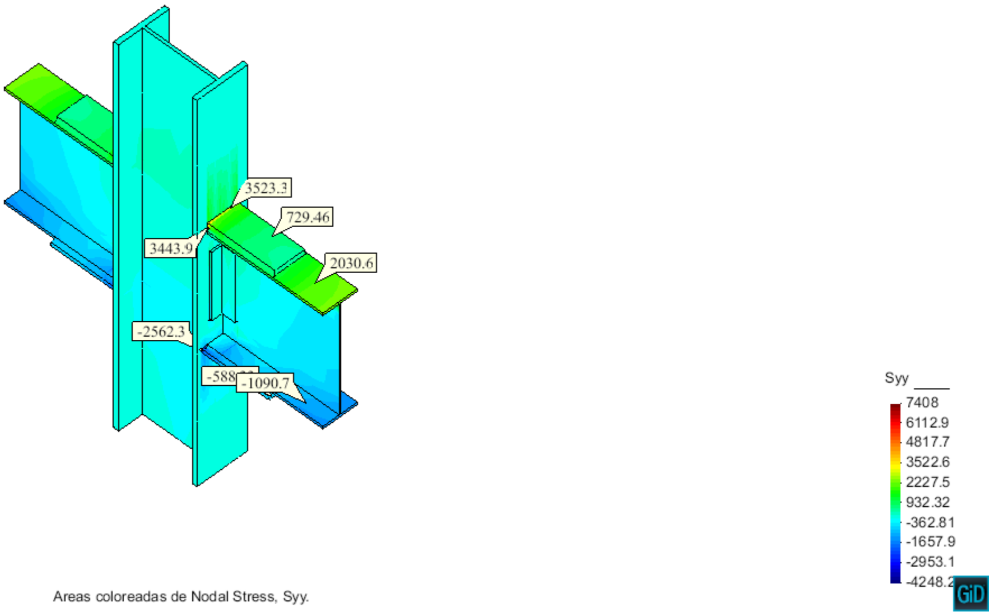


Figure 135: Vertical displacements (Z-axis) in M3-A572 N-QLF model.

Finite Element Analysis results: Model 3, Tension stresses S_{yy}.



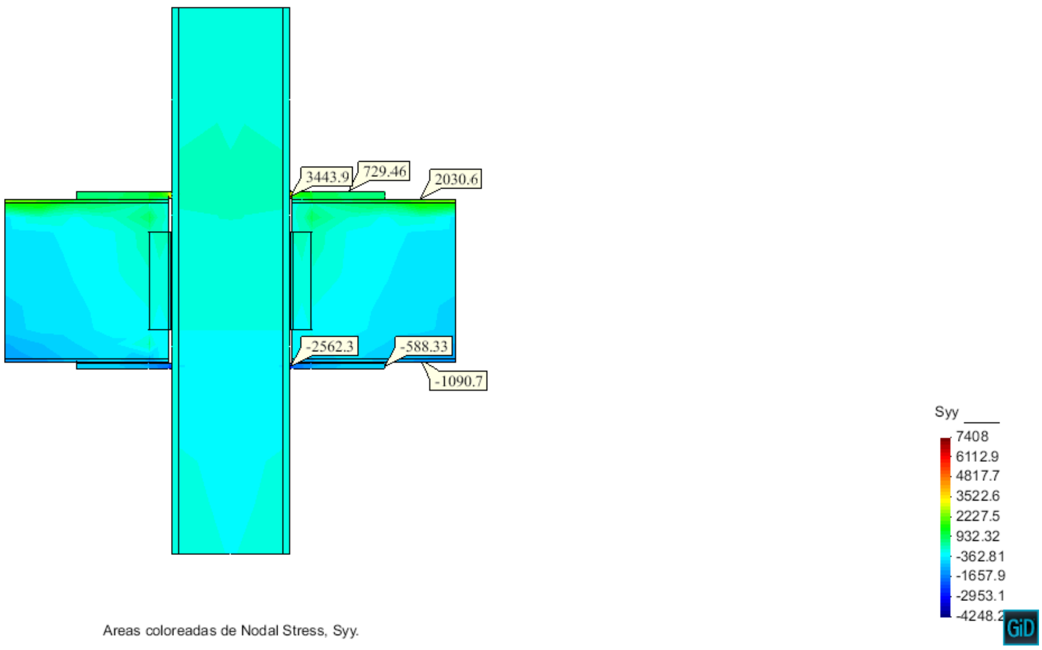


Figure 136: Axial stresses (Y-axis, SYY) in M3-A572 N-QLF model.

Observations:

- The results of tensile stress for upper plate barely exceed yield stress with a very small difference.
- The compression plate have a good performance with a demand even lesser that the tension plate.

Finite Element Analysis results: Model 3, Shear stresses S_{xy}.

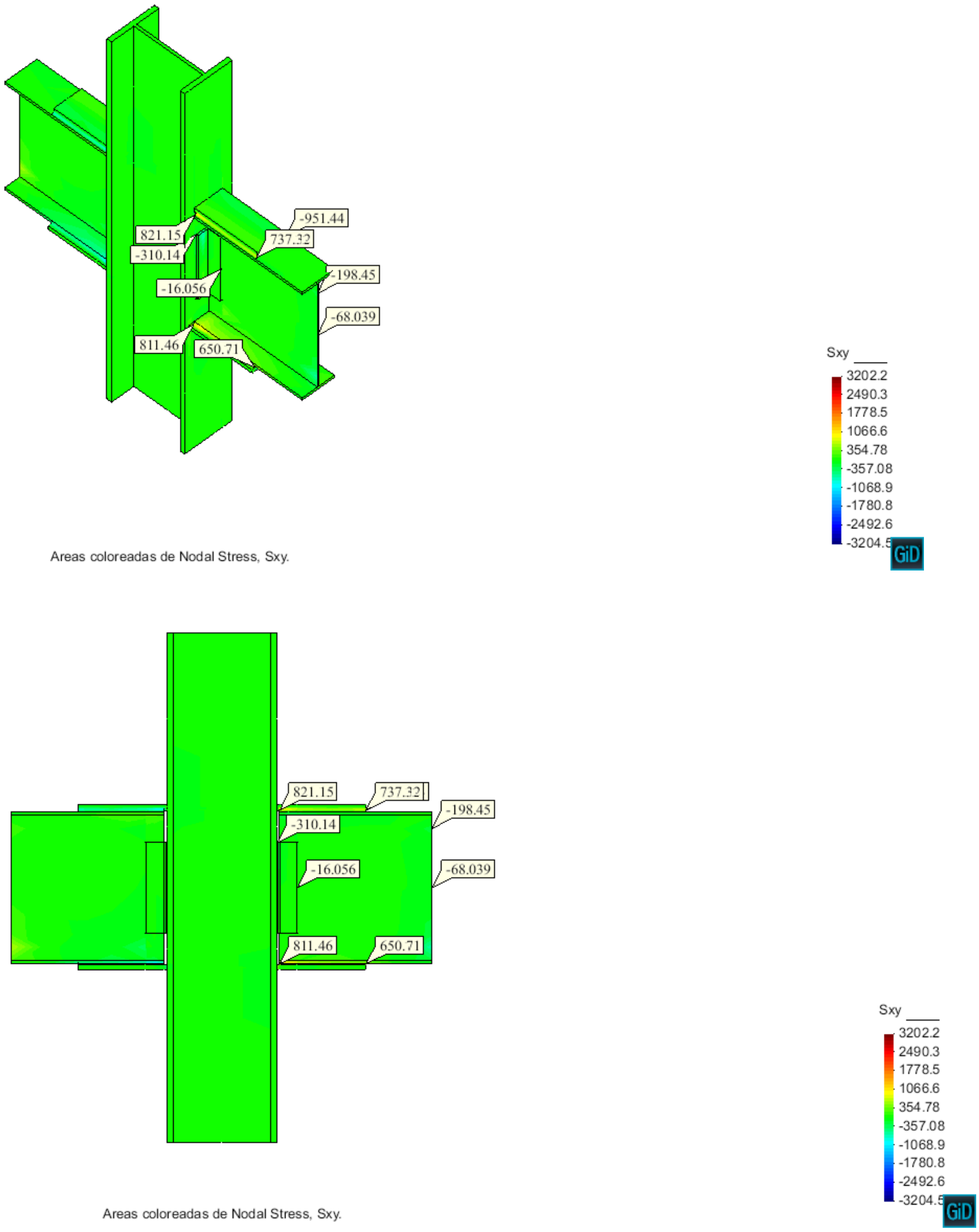


Figure 137: Shear stresses (XY plane, SXY) in M3-A572 N-QLF model.

Observations:

- All values show an acceptable development.

Finite Element Analysis results: Model 3, Shear stresses S_{yz}.

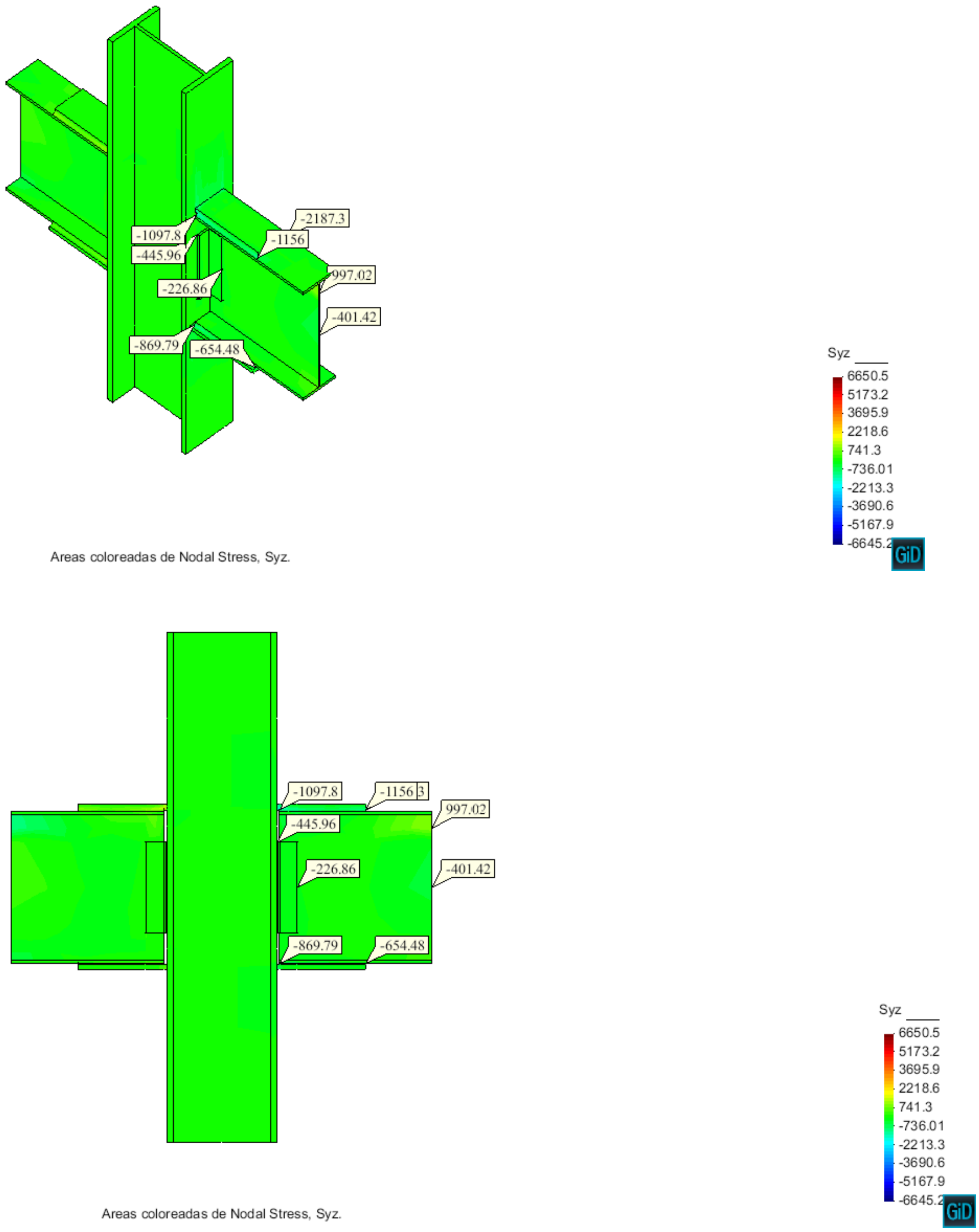


Figure 138: Shear stresses (YZ plane, SYZ) in M3-A572 N-QLF model.

Observations:

- Again, those regions inside the gaps of upper plates have an excess of shear stress. The rest of values are allowable.

Finite Element Analysis results: Model 3, Von Mises stresses VM_s.

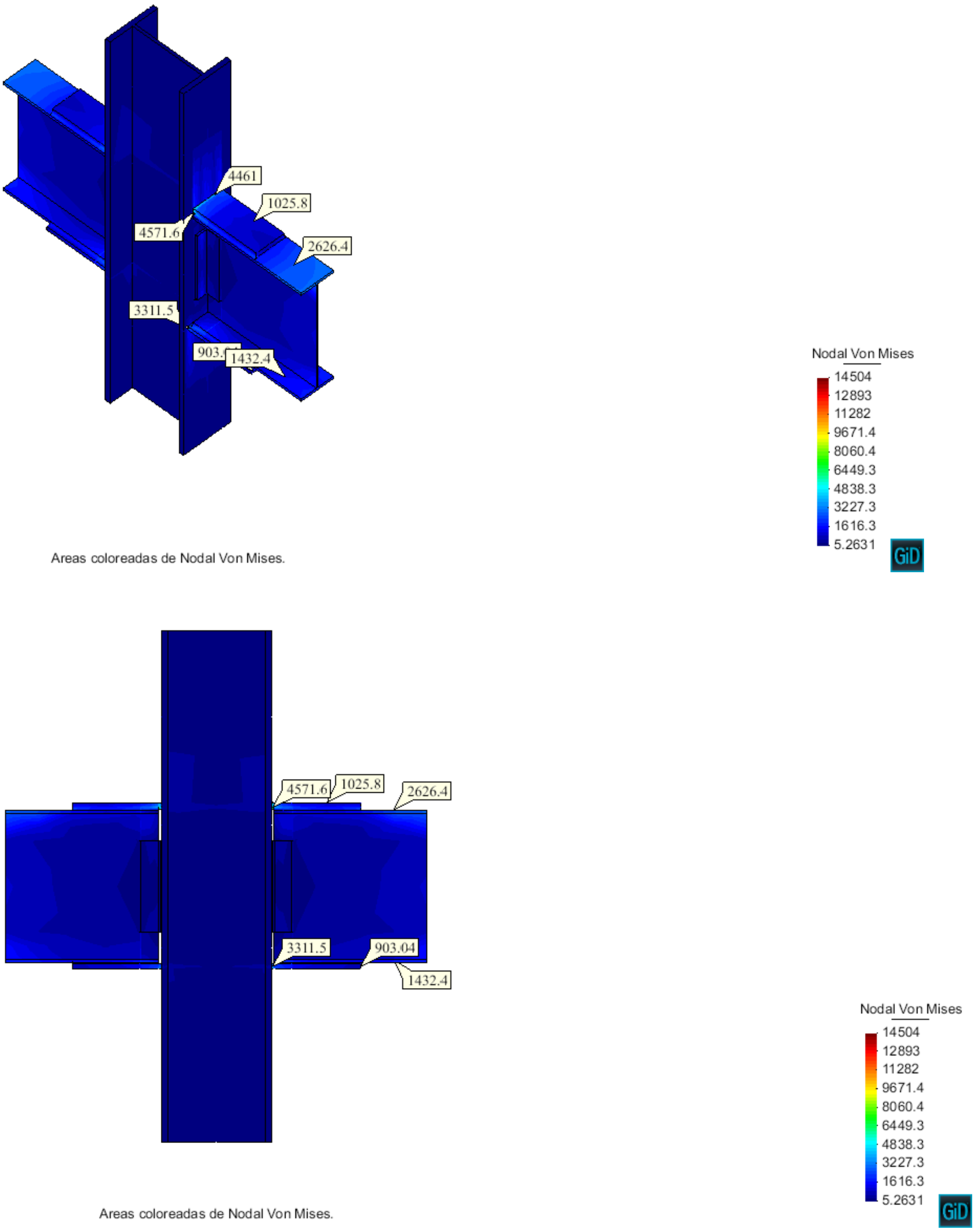


Figure 139: Von Mises stresses in M3-A572 N-QLF model.

Observations:

- The U-shape pattern along the web of the column is presented only for the zone of tension plates connected, equally as the previous models. Comparing these last configurations respect to the models with ASTM A36 union plates, this pattern is shown for upper and lower areas.
- The flanges of the beam do not form representative yield surfaces.

8 Conclusions and Future Research Work

Along the whole work and research developed, it is recognized there is still a big span of ambiguity and uncertainty about results obtained and considering the small group of configurations. Many parameters are involved for this particular investigation, the boundary conditions and computational scope defines very specific premises to reach success and a coherent validation of output data.
Synthesizing the output information, the results of interest are shown below and discussed respectively.

8.1 Steel Connections and ASTM A36 Plates

Conventional configurations (forces taken by plates, weight and rotation of connection)

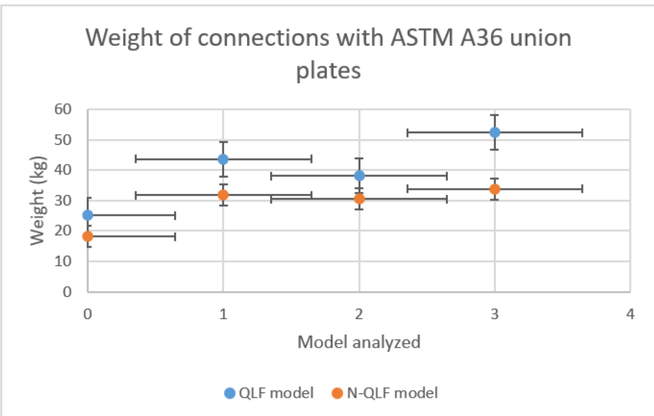
Model	$F_{TP}(kg \cdot f)$	$F_{CP}(kg \cdot f)$	**Weight (kg)**	**Rotation (θ, rad)**
M0	1.03e+05	8.80e+04	25.03	0.001324
M1	1.35e+05	1.49e+05	43.64	0.001264
M2	1.22e+05	1.23e+05	38.10	0.000997
M3	1.46e+05	1.48+05	52.34	0.000872

Non-qualified configurations (forces taken by plates, weight and rotation of connection)

Model	$F_{TP}(kg \cdot f)$	$F_{CP}(kg \cdot f)$	**Weight (kg)**	**Rotation (θ, rad)**
M0	8.20e+04	8.90e+04	18.12	0.001100
M1	1.12e+05	1.02e+05	31.74	0.001126
M2	1.13e+05	9.80e+04	30.74	0.000770
M3	1.12e+05	1.02+05	33.78	0.000850

Notation:

- F_{TP}: upper plate subjected to tension.
- F_{TP}: lower plate subjected to compression.



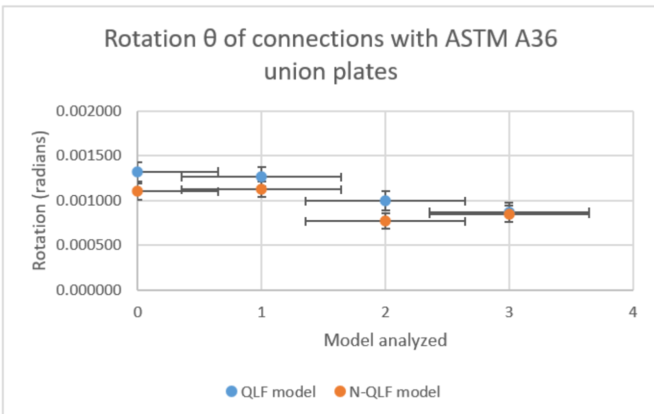


Figure 140: Left graphic depicts the comparison of the weight of connection for every model analyzed, and to the right the correspondent rotation in radians.

Performing a detailed contrast between the results obtained and having a clear visualization trough last graphics, there is a summary of weight decreasing comparing the conventional model respect to angles-coupled. Also, it is shown the reduction percentage quantity comparing both cases.

Models compared	**Weight reduction/difference ratio (%)**	**Rotation θ reduction/difference ratio(%)**
M0-CONV/M0-ATYP	27.60	16.9
M1-CONV/M1-ATYP	27.30	11.0
M2-CONV/M2-ATYP	19.80	22.7
M3-CONV/M3-ATYP	35.50	2.6

Analysing data, there is an approximate average percentage weight optimization of using the non-qualified connection about 27.54%. Also, it is notable that rotations for angles-coupled connections are smaller than presented in conventional models with an average percentage reduction or an increasing restriction capacity to rotate about 13.30%.
According to gotten results, there is logic in concluding that non-qualified set has features very similar to a *Fully Restrained (FR)* or *Type 1* moment connection.

8.2 Steel Connections and ASTM A572 Gr. 50 Plates

Conventional configurations (forces taken by plates, weight and rotation of connection)

Model	$F_{TP}(kg \cdot f)$	$F_{CP}(kg \cdot f)$	**Weight (kg)**	**Rotation (θ, rad)**
M0	1.00e+05	1.19e+05	19.49	0.001430
M1	1.56e+05	1.35e+05	34.05	0.001429
M2	1.31e+05	1.30e+05	30.64	0.001111
M3	1.56e+05	1.54+05	42.24	0.000962

Non-qualified configurations (forces taken by plates, weight and rotation of connection)

Model	$F_{TP}(kg \cdot f)$	$F_{CP}(kg \cdot f)$	**Weight (kg)**	**Rotation (θ, rad)**
M0	8.60e+04	1.22e+05	16.56	0.001056
M1	1.09e+05	1.15e+05	26.23	0.001283
M2	1.05e+05	9.50e+04	22.89	0.000983
M3	1.25e+05	1.15+05	29.07	0.000920

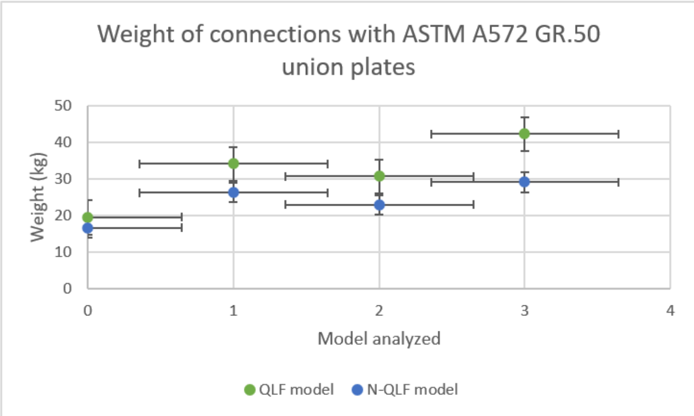
Notation:

- F_{TP}: upper plate subjected to tension.
- F_{TP}: lower plate subjected to compression.

Performing a detailed contrast between the results obtained and having a clear visualization trough last graphics, there is a summary of weight decreasing comparing the conventional model respect to angles-coupled set. Also is shown the reduction percentage quantity comparing both cases.

Models compared	**Weight reduction/difference ratio (%)**	**Rotation θ reduction/difference ratio(%)**
M0-CONV/M0-ATYP	15.0	26.10
M1-CONV/M1-ATYP	23.0	10.3
M2-CONV/M2-ATYP	25.3	11.5
M3-CONV/M3-ATYP	31.2	4.4

Comments:



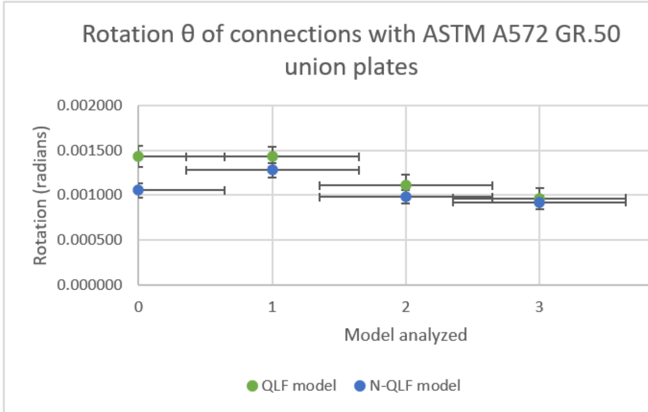


Figure 141: Left graphic depicts the comparison of the weight of connection for every model analyzed, and to the right the correspondent rotation in radians.

- Average weight reduction ratio: 23.62%.
- Average rotation restriction increasing ratio: 13.08%.
- The set of angles instead of shear plates reduce the values of tensions concentrated inside the gaps between the the angle legs and the flanges of column (both cases, union plates with designation A36 and A572 Gr. 50).

8.3 General Conclusions

- All connections showed certain stress congestions in the extremes of fillet welds between gaps under bending plates and among shear/angles connected to column flanges subjected to shear. This behavior could be conduct to a possible detachment for welds located at extreme edges of fillets connected to shear plates/angles.
- Fillets connected to angles-coupled can be resisted according to the features of the weld.
- The connection M1 must be either redesigned for every configuration or providing stiffener/girder plates.
- Despite the classic methodology of computing is not a very precise tool (section 3.5 of this research document), this procedure is still a good reference as start point to dimension properties for rigid steel connections. Realizing not every case of study accomplished the objectives of design, the couple of methodologies exposed in this thesis can not be dismissed at all, and the approach by finite element method or experimental tests can be adjusted.
- A reliable design factor to support the rigid design can be considered a figure not greater of the 80% of the yield bending moment.
- Non-qualified connections present a rare pattern with a U-shape of tensional distribution affecting the web and flanges of the column, compared to conventional models that such concentration dissipates principally over the web of the column.
- The axial stresses (S_{yy}) are increased approximately a 20% along the edges connected with the fillet welds with the flanges of the beam and column. This response can be caused by the shear lag effect.
- A failure plane for welds can be observed over the body of the fillet with an inclination angle of 45º.
- Beyond that classic rigid design method applied for non-qualified steel connections consider a less force transference, the results for prototypes subjected to such methodology exhibit a greater rotational stiffness.

- According to the contrast of weight connections table, all specimens with ASTM-A36 steel designation of union plates require on average 27.5% less material than traditional prototypes. Analogously, samples studied to validate with ASTM A572 Gr. 50 employ a 23.62% less material than conventional.

8.4 Future Scope of Research

After a punctual research, it is worth that this project continue a certain development in future. The new openings and premises included might be estimated are the following:

- Construct a greater field of combinations for different connections and configurations with several properties to make a study more complete with statistical indicators more representative and reliable. Some examples could be either models with welded joints but adding a stiffener/girder plate in a strategic location between bending plates and the flanges of the column or using variable width diaphragm with the greater width welded over the column flanges.

- Evaluate the methodology of design for bolted connections, and specially, trying to replicate and validating the sets shown in this document with the purpose of inducing flexure trough bolts employed.

- Study the mechanic response and performance of the steel connection now considering the contribution of continuity plates located at the web and flanges of the column.

- Refine the methods to dimension and structural solutions of steel connections to provide resistant and safety arrays.

- Pursuing the implementation of real models to validate results trough experimental engineering in material labs.

- Highlight the importance of studying steel connections trough punctual and detailed investigation and experimental developments.

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