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Authors:	Jean Claude Gerardy, Structural Engineer, ArcelorMittal Georges Axmann, Head of Technical Advisory, ArcelorMittal Riccardo Zanon, Structural Engineer, ArcelorMittal André Plumier, Professor, ArcelorMittal
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Design of Composite Mega-Columns with Multiple Rolled H-Sections
带有多个轧制H型构件的组合巨型柱的设计

Jean-Claude Gerardy

Georges Axmann

Riccardo Zanon

André Plumier

Jean-Claude Gerardy, Georges Axmann & Riccardo Zanon

ArcelorMittal Commercial Sections
66, rue du Luxembourg,
Esch-sur-Alzette, L-4009, Luxembourg

tel (电话): +352 5313 2616, +352 5313 2654, +352 5313 3031
email (电子邮箱): jc.gerardy@arcelormittal.com; georges.axmann@arcelormittal.com; riccardo.zanon@arcelormittal.com
www.arcelormittal.com/sections

Jean-Claude Gerardy is a structural engineer responsible for the worldwide promotion, marketing and sales of ArcelorMittal sections in tall buildings. He holds a Master Degree from the University of Liege in Belgium.

Jean-Claude Gerardy是结构工程师，安赛乐米塔尔公司负责全球超高层项目中的型材市场推广和销售。获得比利时Liege大学硕士学位。

Georges Axmann is head of technical advisory and marketing for sections & merchant bars at ArcelorMittal Long Carbon Europe.

Georges Axmann是安赛乐米塔尔公司欧洲，碳钢型材及棒材的技术咨询及市场部负责人。

Riccardo Zanon is a structural engineer in the technical advisory and marketing for sections & merchant bars at ArcelorMittal Long Carbon Europe.

Riccardo Zanons是安赛乐米塔尔公司欧洲长材部门，市场及技术支持部门的结构工程师。

André Plumier

Plumiecs & ULg
BAT B52/3 Allée des Chevreuils,
Liege, B-4000, Belgium

tel (电话): +32 4 3669230
email (电子邮箱): a.plumier@ulg.ac.be
www.arcelormittal.com/sections

André Plumier is a professor at the University of Liege (Belgium), with specialties in steel and composite steel-concrete structures and seismic design. He was the inventor of the reduced beam sections concept. He has beenfull member of the ECCS TC13 seismic design of steel structures committee since its creation.

André Plumier是比利时Liege大学教授，专长于钢及钢砼复合结构以及抗震设计的研究，同时也是狗骨钢梁概念的创始人。并从那时开始成为了ECCS TC13 钢结构抗震设计委员会成员。

Abstract

Composite mega columns of tall buildings are currently designed with continuous built-up sections, welded in the fabrication shop and spliced on the job site without any prequalified welding procedure. This leads to highly restrained welds and splices which, under severe dynamic loadings, will likely crack before exhibiting any ductile behavior. The 1994 earthquake in Northridge, California, taught us that welding procedures, beam-to-column connections, and column splices have to be as simple as possible to work properly and reliably. Using multiple rolled sections encased into concrete is the solution for increasing the safety of tall buildings. It leads to less welding, less fabrication works and reliable, simple splices which have been used for decades in high-rise projects. AISC allows engineers to design composite sections butdoesn't explain how to perform and check the design; this paper offers a method to do it.

Keywords: Composite Columns, Rolled Sections, Steel Shapes, Mega-Columns

摘要

超高层的复合截面巨柱中的型钢加劲，目前多数设计为组合截面型钢，在加工厂焊接，工地现场拼接，并且没有合格的焊接流程。这将导致拼接处较高的焊接应力集中，在剧烈的动载荷作用下，钢材可能在延性发挥之前发生就开裂。1994年美国加州北岭大地震让我们认识到，梁柱连接，柱子的拼接节点的焊缝要尽可能的简单，以期可以可靠地工作。 使用多个型钢加劲混凝土柱子的方案可以提高超高层的安全性能。 可以减少焊接量、加工量， 而可靠而且简单的拼接在过去的几十年里已经成熟应用在超高层的项目中了。美国钢结构协会允许工程师设计复合截面柱，但是没有解释怎样具体执行和验算，本文将就此理念给出一个设计和检验的方法。

关键词： 复合截面柱，热轧型钢，超高层，设计方法，巨柱

Introduction

Mega composite columns of tall buildings in Asia are typically designed with huge steel continuous caissons built-up from heavy plates. They are welded together in the steel fabrication shop and spliced on the job site (see Figure 1).

Internationally recognized welding codes such as AWS D1.1 (structural American welding code) and AWS D1.8 (seismic welding code) or EN 1090-2:2008 (execution of steel structures) and EN 1011-2:2001 (recommendations for welding of metallic materials) impose the pre-qualification of the welding procedures of such "exotic" joints, following strict welding sequences. Required preheating and interpass temperatures are specified per the thickness of the steel (>32mm), its composition (CEV/grade), the type of electrode and the level of restraint in the joint. Non-destructive tests (ultrasonic test, magnetic particle examination, radiographic test) performed by certified inspectors are mandatory to guarantee sound welded connections and a safe structure.

In practice, even when the welding codes are

引言

在亚洲，超高层巨柱目前普遍设计成了厚板焊接而成的巨大的实腹钢箱，在加工厂焊接而成，到现场拼接。（见图1）。

全球广泛认可的焊接规范如 AWS D1.1（美国焊接结构规范）及AWS D1.8 （抗震焊接规范）或 EN 1090-2:2008 （欧洲钢结构规范焊接执行标准）和EN 1011-2:2001（金属材料焊接规程）都实行了焊接工艺的资格预审，比如"异型"节点，按照严格的焊接顺序。规定了钢材不同厚度（>32mm）、碳当量值所需要的预热和层间温度，焊药的等级及节点应力集中程度。有认证的检验员对焊接进行强制性的非破坏性检测（超声波探伤，磁粉检验，放射线探伤）以确保可靠的焊缝和结构安全。

在项目实践中，即使是严格按照焊接规范实施焊接，一些简单的结构通常还是会有10%的返修率。

就钢箱结构而言，焊接条件就更恶劣。普通50级金属（ASTM A572Gr.50或Q345）的厚板焊接必须在加工厂或现场，于整个焊接过程中都要预热到110° C以上。 在焊接这些巨大的箱体时，任何的预热的缺失都会引起材料的敏感（冷脆）和各个方向的较高的焊接应力，从工厂加工开始并到

strictly followed, it is typical to have to repair up to 10% of the welds in simple structures.

In the case of these huge caissons, the welding conditions are rather extreme. Heavy thick plates in typical grade 50 steel (ASTM A572Gr.50 or Q345) must be preheated at 110°C in the steel fabrication shop as well as on the job site prior and during the welding process. Any lack of preheating when welding these huge caissons induces sensitive material conditions (hard and brittle zones) and high levels of restraint (post weld stresses) in all directions starting in the steel fabrication shop and amplified on the job site after splicing two caissons together. Applying adequate preheating during the whole welding process is difficult. How to preheat such joints at 100°C? Correct welding takes days of work without interruption. Proper controlling and repair of all welds is so expensive that this solution, when correctly executed, is not economical at all.

There is an economical and safer alternate to this configuration. AISC design codes allows designers to use composite sections built-up from two or more encased steel shapes provided that the buckling of individual shapes is prevented before the hardening of the concrete.

The Chinese Institute of Earthquake Engineering is also recommending the use of multiple jumbo H-shapes rather than large continuous caissons. The welding procedures and the connection detailing of single rolled-H-sections are well described in the above mentioned codes. The use of correct beveling, the so-called "weld-access-holes" (see Figure 2). associated to very precise welding sequences, including the removal of the backing bars, and appropriate grindings to clean-up the weld surface between passes minimize the amount of residual stresses after splicing single rolled steel columns. W14x16 (HD400) rolled sections (jumbos) are today available up to 1299 kg/m (873 lbs/ft) with a flange thickness of 140 mm (5.5 in.) and W36 (HL920) are available up to 1377 kg/m (925 lbs/ft). These sizes are not only available in classical grade 345 MPa (ASTM A992/Grade 345, Q345, S355) which requires them to be preheated for flange thicknesses above 32 mm (1.5 in.) but also in high tensile modern steel produced by a quenching and self tempering process, namely ASTM A913 Grade 345 and 450, or per ETA 10-156 (European Technical Approval) grades Histar 355 and Histar 460. Besides their higher yields, the main advantages of these high performance steels are their weldability without preheating (above 0°C and with low hydrogen electrodes) as well as their outstanding toughness. (27J up to minus 50°C). These high performance steels are not only fully in compliance with American and European standards, they can also meet the stringent requirements of the Chinese standards, such as the 20% minimum elongation which is mandatory in the Chinese seismic codes. These QST steels (ASTM A913) have already been successfully used in the Shanghai World Financial Center.

The method of designing multiple H sections encased into a composite column is unfortunately not described in any designing structural book. But it can be found for free at http://www.arcelormittal.com/sections

Design Example of Composite Mega Column with 4H-Rolled Sections

In order to demonstrate the practical application to a modern composite mega-column, a section with outer dimensions about 3 x 3m has been considered, with four jumbo sections encased in the corners (see Figure 3). The steel grade chosen is Grade 65 ksi (450MPa) according ASTM A913-11, concrete grade C50/60. The cross-section

Figure 1. Continuous caissons (Source: Shanghai Center)
图1. 连续沉箱（出自：上海中心）

Figure 2. Column splicing Jumbo beveling (Source: ArcelorMittal)
图2. 巨型斜角拼接而成的柱子（出自：安赛乐米塔尔）

现场2个箱体的拼接。在整个焊接过程中给予充分的预热是相当困难的，但不是不可能。怎样将这样的连接节点预热至110° C ? 正确的焊接需要几天的时间并且无间断。这样的方案，对所有这些焊缝合理的控制和修补的成本是很高的，如果完全按照正确的方式履行焊接是完全不经济的。

对于这样的构造，现在有一种经济又安全的替代方案。美国钢结构协会在设计复合截面柱方面允许使用2个或者2个以上外包型钢的组合型钢加劲，从而避免在混凝土硬化前各个部分发生变形。

中国抗震审查委员会也推荐，使用多个H型钢加劲形式加劲优于大型实腹式钢箱。单个H型钢的焊接工艺和节点做法都已经在之前提到的相关规范中详述。使用正确的切边，即所说的“焊接引入孔”（见图2）关系到非常精准的焊接顺序、打磨清理每道焊缝表面，以将单个型钢拼接残余应力将至最低。W14x16（h/b接近1，HL920）系列现在已经有了每延米重1299kg/m，翼缘厚度可达144mm。W36（h/b大于1，厚翼缘、腹板）系列每米可达1377kg/m。这些规格不仅有传统材质Q345，其要求厚度超过32mm即需要预热；还有通过淬火-回火工艺轧制出来的高强度钢材，即ASTM中的A913，屈服强度从345MPa~450MPa。在具有高强度的同时，A913最显著的性能是良好的可焊性，即0° C以上无需预热；并且居然有很好的延性和韧性（-50° C27J）。A913的性能不仅完全与美国和欧洲规范一致，并且可以满足中国抗震规范规定的钢材延伸率20%的要求，对于高强度钢材来讲，这是一个比较严苛的规定。经过淬火—回火轧制的钢材（ASTM A913）已经成功应用于上海环球金融中心等超高层项目。

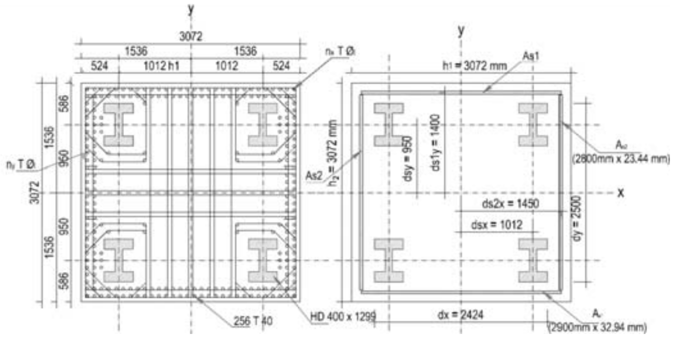


Figure 3. Design example of composite mega-column with 4 rolled sections (Source: Plumiecs & ULg)
图3. 带有4个热轧型钢的复合截面巨柱设计案例（出自：Plumiecs & ULg）

is reinforced with longitudinal bars, transverse struts and ties (with nominal yield strength of 500 MPa).

Combined Flexure and Axial Force

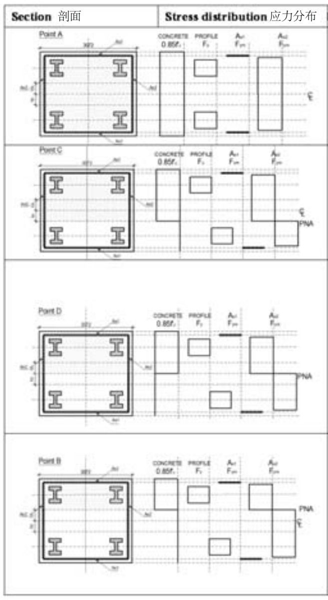
The interaction between axial forces and flexure in composite members is governed by AISC Specification Section I5 which, for compact members, permits the use of a strain compatibility method or plastic stress distribution method, with the option to use the interaction equations of Section H1.1. The strain compatibility method is a generalized approach that allows for the construction of an interaction diagram to follow the same principle used for reinforced concrete design. Application of the strain compatibility method is required for irregular/nonsymmetrical sections. Plastic stress distribution methods are discussed in AISC Specification Commentary Section I5 which provides three acceptable procedures for filled members. The method to determine the interaction curve is an approximation based on polygonal interpolations that allows the use of less conservative interaction equations than those presented in Chapter H. The interaction curves which will be used in this paper correspond to the second approach (see Figures 4 and 5), 4 - N-M Interaction curve.

The equations correspond to different points selected on the interaction curves. Calculations concerning the slenderness effect are not presented, because they would not be different from those shown in detail in AISC (2011). In the plastic-distribution method, the N-M interaction curves are convex, because it is assumed that the concrete has no tensile strength. For a composite cross-section symmetrical about the axis of bending, Roik and Bergmann (1992) have proposed a simple method to evaluate its M-N interaction diagram. This method is adopted in AISC Specifications (2010). As shown in the figure below, this method does not determine a continuous N-M interaction curve, but only a few key points.

The M-N curve is then constructed by joining these key points by straight lines. When evaluating these key points, rigid-plastic material behavior is assumed. Thus, steel is assumed to have reached yield in either tension or compression.

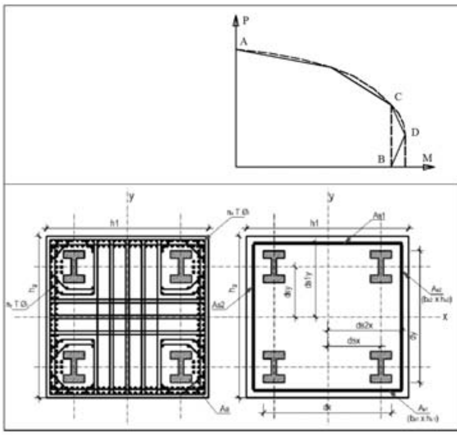
The key points in Figure 5 are:

- A – squash load point
- B – pure flexural bending point
- D – the maximum bending moment point
- C – point with bending moment equal to the pure bending moment capacity



Section 剖面	Stress distribution 应力分布	Point 点	Defining Equations 定义方程
		A	$P_A = A_s \cdot F_y + A_{s1} \cdot F_{ysr} + A_{s2} \cdot F_{ysr} + 0.85 \cdot A_c \cdot f'_c$ $M_A = 0$ A_s = area of the steel shape $A_C = h_1 \cdot h_2 - A_s - A_{sr}$
		C	$P_C = 0.85 \cdot A_c \cdot f'_c$ $M_C = M_B$
		D	$P_D = \frac{0.85 \cdot A_c \cdot f'_c}{2}$ $M_{Dx} = Z_{sx} \cdot F_y + (Z_{r1x} + Z_{r2x}) \cdot F_{ysr} + \frac{1}{2} \cdot Z_{cx} \cdot (0.85 \cdot f'_c) Z_{cx}$ $Z_{r1x} = 2 \cdot A_{s1} \cdot d_{s1y}$ $Z_{r2x} = 2 \frac{b_{s2} \cdot h_{s2}^2}{4}$ $Z_{sx} = 4 \cdot A_s \cdot d_{sy}$ $Z_{cx} = \frac{h_1^2}{4} - Z_{r1x} - Z_{r2x} - Z_{sx}$
		B	$M_{Bx} = M_{Dx} - Z_{r2xn} - \frac{1}{2} \cdot Z_{cxn} \cdot (0.85 \cdot f'_c)$ $Z_{cxn} = h_1 \cdot h_{nx}^2 - Z_{r2xn}$ For h_{nx} between the two profiles $(h_{nx} \le d_{sy})$: $h_{nx} = \frac{P_C}{2 \cdot h_1 \cdot (0.85 \cdot f'_c) + 8 \cdot b_{s2} \cdot F_{yrs}}$ $Z_{r2xn} = 2 \cdot b_{s2} \cdot h_{nx}^2$ Z_{cxn} - x-axis plastic modulus of concrete section Z_{r2xn} - x-axis plastic modulus of A_{s2} plate

Figure 4. N-M Interaction curve (Source: Plumiecs & ULg)
图4. N-M曲线 （出自：Plumiecs & ULg）



$A_{s1} = n_x \cdot A_{sr1} = b_{s1} \cdot h_{s1}$
$A_{s2} = n_y \cdot A_{sr2} = b_{s2} \cdot h_{s2}$
$A_{sr} = (n_x + n_y) \cdot A_{sri}$
n_x –no. of bars on x direction
n_y – no. of bars on y direction
b_{s1}= width of A_{s1} plate
h_{s1}= height of A_{s1} plate
b_{s2}= width of A_{s2} plate
h_{s2}= height of A_{s2} plate
A_{s1} = area of top (bottom) plate
A_{s2} = area of lateral plate
A_{sri} = area of one longitudinal bar

Figure 5. N-M Interaction curve (Source: Plumiecs & ULg)
图5. N-M曲线 （出自：Plumiecs & ULg）

使用多个型钢加劲混凝土复合截面巨柱的设计方法至今还没有在任何一本结构设计书籍中介绍过，在此与大家分享。带有详细计算过程的完整文件请免费下载：http://www.arcelormittal.com/sections

由4个热轧H型钢加劲的复合截面巨柱设计示例

为显示在巨柱中实际应用，现以外包尺寸为3mx3m的复合截面柱为例，角部以4个轧制H型钢加劲（见图3）。钢材强度等级根据ASTMA913-11选用450MPa，砼强度C50/C60. 截面由纵向钢筋、水平栓钉及箍筋（通常选用屈服强度500MPa）加强。

压弯组合

复合截面的压弯作用在AISC之I5中阐述，对于紧凑型截面构件，允许使用应变协调法或塑性应力分布法，即H1.1章的作用力公式。应变协调法是与钢筋砼结构设计方法一致的基本方法，得出作用力的曲线。一般几何尺寸不对称、不规则是采用应变协调法。塑性应力分布法在AISC之I5中阐述，为填充构件提供了3种可用的工艺。决定作用力曲线的方法是一种近视法，基于多边形的插补，较少使用H章提及的保守的力学计算公式。本文中的作

Transversal Shear

According to AISC Specification Section I4.1, there are three acceptable options for determining the available shear strength of an encased composite member:

- Option 1- Available shear strength of the steel section alone in accordance with AISC Specification Chapter G.
- Option 2- Available shear strength of the reinforced concrete portion alone per ACI 318.
- Option 3- Available shear strength of the steel section in addition to the reinforcing steel ignoring the contribution of the concrete.

Option 1 clearly is a gross underestimation for the section with four encased steel profiles, because it would consist in disregarding the contribution to shear resistance of the important concrete area.

Option 2 is recommended. However, its application requires one adaptation for composite sections with several encased steel profiles, in comparison to, for instance, the procedure presented in Design Example I.11 (AISC 2011). It requires the separate calculation of shear strength of sub-sections composing the complete section. The problem to solve in sections with several encased profiles is that concrete and steel components contributing to shear resistance are not working in parallel, as in the case of one central steel profile encased in concrete: they are, for some part, working in a series or "chain". This is easier to understand if one subdivides the column section into five smaller sections, each providing resistance to shear. The whole procedure is presented in detail in Plumier et al. (2012).

Option 3 is not recommended because it could be unsafe for composite sections with several encased steel profiles. In fact, the available shear strength would be found as the addition of the available shear strength of the steel sections in addition to the available shear strength of the reinforcing steel, ignoring the contribution of the concrete. This does not express clearly what is meant. The idea is that, due to cracking, the contribution to shear resistance of concrete without transverse reinforcement is equal to 0. In such case, the shear resistance of reinforced concrete in shear can exist, due to transverse reinforcement and equilibrium between inclined compression struts of concrete and tension in steel "ties," the stirrups. However, in a section with several steel profiles, the applied shear force will distribute itself proportionally to the stiffness of those sections. Section being made of components working in series or "chain," the strength of the chain should be calculated on the basis of its weakest link, which is concrete. So Option 3 has to be applied in the same way as Option 2 and adding the shear strength of the steel profiles to shear strength of the "reinforcement" would lead to an unsafe design.

Longitudinal Shear

The calculation of the acting longitudinal shear is analogous to any other composite section. For the given example see Plumier et al. (2012). In order to resist the given shear there are two main options:

- Shear connectors, which may be headed stud anchors (see Figure 6).
- Direct bearing, which may be created for example through transverse stiffeners (see Figure 7).

In cases where longitudinal shear is small (columns basically subjected to pure axial loading), available surface bond may be sufficient.

用力曲线与第一种方法一致。（见图4和图5）4-N-M图。

公式所对应的曲线上的不同点，计算中考虑了长细比的作用，由于其与AISC 2011 中的部分相同不再赘述。在塑形应力分布法中，N-M曲线是凸的，因为假设砼没有抗拉强度。对于对称的复合截面的弯曲轴，Roik and Bergmann 已经提出了一个简单的方法估算它的N-M曲线。这个方法收录在AISC规范中。如下图所示，此法不能给出连续的曲线，仅仅是几个关键点。

那么M-N曲线就是由连接这些点的直线组成。在推算这些点时，假设材料是钢塑形的。因此，钢材在受拉或受压时达到屈服点。

几个关键点见图5：

- A—受压点
- B—纯弯点
- C—最大弯矩点
- D—弯矩与纯弯矩承载点

横向剪力

依照AISC之I4.1，有三种许用方法确定劲性构件的抗剪承载力。

- 方法1—按照AISC之G章得出钢骨的抗剪承载力
- 方法2—按照ACI 318 计算劲性钢筋混凝土部分抗剪承载力
- 方法3—忽略混凝土贡献，钢骨与钢筋的抗剪承载力

方法1， 显然低估了有四个型钢加劲的截面，因为它忽视了重要的混凝土的面积对抗剪承载力的贡献。

方法2，推荐使用。但是要求复合截面有多个型钢加劲，有关比较，计算过程在设计示例I.11（AISC 2011）。此法要求分别计算几个型钢截面的抗剪强度集合为整个截面的抗剪强度。这里要解决的问题是截面的几个加劲钢骨与混凝土的抗剪能力不是平行工作的，好比只有在柱中心有一个钢骨的情况，在某些地方，它是以系列的或者“锁链”的形式工作的。这就比较容易理解，如果将一个钢骨再细分为5个小钢骨，则他们分别贡献抗剪能力。整个分析计算步骤详见Plumier et al. (2012)。

方法3，不推荐，因为作为多个钢骨加劲的复合截面它会是不安全的。实际上，有效的抗剪承载力是除了钢筋以外钢骨所提供剪力，忽略了混凝土的贡献。这不能清楚的表达真正的含义。这一观点是说由于裂缝，没有横向加劲的混凝土的对抗剪承载力的贡献是0. 这种情况下，钢筋混凝土的抗剪承载力是存在的，由于横向加劲及混凝土斜截面偏压与箍筋中的拉力间的平衡。然而，对于由几个型钢加劲的截面来说，剪力会自动按照型钢刚度分配。由几个构件组成的柱截面就是这样以连续的工作链形式工作。这个工作链以最薄弱的地方为基准代表其强度结算得出，那就是混凝土。所以方法3应该等同方法2一样的应用，并将钢骨的抗剪承载力加之于钢筋的抗剪承载力，这将导致不安全设计。

纵向剪力

对于作用的纵向剪力与其他任何复合截面类似。给出算例见Plumier et al.(2012)。抵抗此剪力有2种途径：

- 抗剪连接件， 这可以是有头的栓钉（见图6）
- 直接抗剪构件，可以是像横向抗剪加劲板（见图7）

在纵向剪力比较小的地方（柱子主要是取决于纯弯曲载荷），已有的表面粘结力可能足够。

由于本文篇幅所限，只能在这里介绍推荐的一部分方法。更完整的审阅请参考Plumier et al.(2012)。

Due to limited place in this paper, only some recommendations will be presented here. For the complete review please make reference to Plumier et al. (2012).

Steel Headed Stud Anchor Detailing Limitations of AISC Specification Sections I6.4a, I8.1 and I8.3

Anchors must be placed on at least two faces of the steel shape in a generally symmetric configuration. That limitation applies at sections with one central encased steel profile. Here the anchors are placed symmetrically with respect to the axis of symmetry of the complete section. There is no reason to have a symmetric configuration for each individual steel profile.

- Maximum anchor diameter: dsa ≤ 2.5t f
- Minimum steel headed stud anchor height-to-diameter ratio: h / dsa ≥ 5
- Minimum lateral clear concrete cover = 25,4mm
- Minimum anchor spacing: smin = 4 dsa
- Maximum anchor spacing: smax= 32 dsa
- Clear cover above the top of the steel headed stud anchors:Minimum clear cover over the top of the steel headed stud anchors is not explicitly specified for steel anchors in composite components; however, in keeping with the intent of AISC Specification Section I1.1, following the cover requirements of ACI 318 Section 7.7 for concrete columns, a clear cover of 38 mm is requested.

Concrete Breakout

AISC Specification Section I8.3a states that in order to use Equation I8-3 for shear strength calculations, concrete breakout strength in shear must not be an applicable limit state.

Direct Bearing

One method of utilizing direct bearing as a load transfer mechanism is through the use of internal bearing plates welded between the flanges of the encased W-shape as indicated in Figure I.X4-7. Where multiple sets of bearing plates are used, it is recommended that the minimum spacing between plates be two times the width of the plates so that concrete compression struts inclined at 45° can develop. That spacing also enhances constructability and concrete consolidation.

Detailing Rules

Material limits are provided in AISC Specification Sections (2010) and I1.3 as follows:

- Concrete strength: $21\text{MPa} \le f'_c \le 70\text{MPa}$
- Specified yield stress of structural steel: $F_y \le 525\text{MPa}$
- Specified yield stress of reinforcing steel: $F_{ysr} \le 525\text{MPa}$

Transverse reinforcement limitations are provided in AISC Specification Section I1.1 (3), I2.1a. (1), I2.1a. (2) and ACI 318 (2008) as follows:

Tie size and spacing limitations

The AISC Specifications requires that either lateral ties or spirals be used for transverse reinforcement. Where lateral ties are used, a minimum of either 10 mm (No. 3) bar placed at a maximum of 406 mm (12 in.) on center, or a 13 mm (No. 4) bar or larger spaced at a maximum of 406 mm (16 in.) on center shall be used.

Note that AISC Specification Section I1.1 (1) specifically excludes the composite column provision of ACI 318 Section 10.13, so it is unnecessary to meet the tie reinforcement provisions of ACI 318

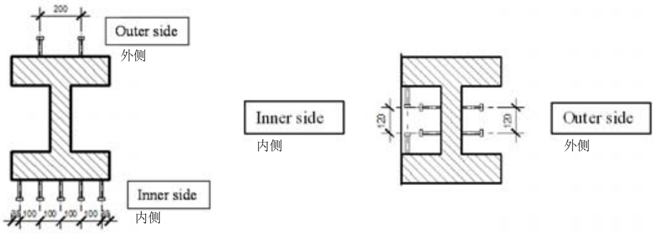


Figure 6. Shear connectors (Source: Plumiecs & ULg)
图6. 抗剪连接件（出自：Plumiecs & ULg）

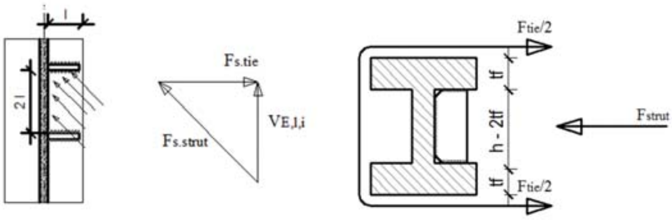


Figure 7. Transverse stiffeners (Source: Plumiecs & ULg)
图7. 横向抗剪加劲板（出自：Plumiecs & ULg）

关于钢拴钉锚固细节的规定--AISC 之I6.4a，I8.1 及I8.3

栓钉必须至少锚固在型材截面对称的2个面上。这个规定应用于仅有一个型钢钢骨的情况。 这里栓钉相对与整个截面的对称轴对称，并不要求每个单个型钢都是对称的外形。

- 最大锚固直径 dsa ≤ 2.5t f
- 最小栓钉锚固高度与直径的比值：h / dsa ≥ 5
- 最小侧面混凝土保护层厚度=25.4mm
- 最小锚固间距：smin = 4 dsa
- 最大锚固间距：smax= 32 dsa
- 栓钉头以上保护层厚度：最小的保护层厚度没有在复合截面构件的规定中明确；但是为了与AISC之I1.1的规定保持一致，根据ACI318之7.7章对于混凝土柱子保护层厚度的规定，要求最小保护层厚度为38mm。

混凝土失效

AISC 之I8.3节规定若用I8-3公式计算抗剪强度，禁止使用混凝土的受剪失效强度作为限值。

直接承载

有一种方法是利用加劲板作为传力机理，通过在2个型钢翼缘之间焊接钢板，见图IX4-7。当使用多个钢板的时候，推荐板间最小间距为2倍的板宽，这样可以将混凝土在45度角方向上的压力传递下去。这个间距还可以为施工带来便利空间以及混凝土的浇筑振捣。

具体的规定

关于材料的限制要求在AISC规定（2010）及I1.3章节如下：

- 混凝土强度： $21\text{MPa} \le f'_c \le 70\text{MPa}$
- 钢材的屈服强度： $F_y \le 525\text{MPa}$
- 钢筋的屈服强度： $F_{ysr} \le 525\text{MPa}$

横向的加劲的限制规定在AISC规定的I1.1 (3), I2.1a. (1), I2.1a. (2) 及 ACI 318 (2008) 如下：

尺寸和间距的限制

AISC规定要求，用箍筋或螺旋筋作为横向的加劲。以箍筋加劲的，最小要么采用10mm的钢筋中心间距最大406mm，或13mm钢筋最大中心间距为406mm。

Section 10.13.8. when designing composite columns using AISC Specifications Chapter I.

If spirals are used, the requirements of ACI 318 Sections 7.10 and 10.9.3 should be met according to the User Note at the end of AISC Specification I2.1a.

Additional tie size limitation

ACI 318 Section 7.10.5.1 requires that all non-pre-stressed bars shall be enclosed by lateral ties, at least 10 mm (No. 3) in size for longitudinal bars 32 mm (No. 10) or smaller, and at least 13 mm (No. 4) in size for 36 mm (No. 11), 43 mm (No. 14), 57 mm (No. 18), and bundled longitudinal bars.

Maximum tie spacing should not exceed 0.5 times the least column dimension

$$s_{max} = 0.5 \cdot \min \begin{Bmatrix} h_1 = 3072\text{mm} \\ h_2 = 3072\text{mm} \end{Bmatrix} = 1536\text{mm}$$

Concrete cover

ACI 318 Section 7.7 contains concrete cover requirements. For concrete not exposed to weather or in contact with ground, the required cover for column ties is 38 mm (1.5 in).

Provide ties as required for lateral support of longitudinal bars

AISC Specification Commentary Section I2.1a references Chapter 7 of ACI 318 for additional transverse tie requirements. In accordance with ACI 318 Section 7.10.5.3 and Fig. R7.10.5, ties are required to support longitudinal bars located farther than 6 in. clear on each side from a laterally supported bar. For corner bars, support is typically provided by the main perimeter ties.

Longitudinal and structural steel reinforcement's limits are provided in AISC Specification Section I1.1 (4), I2.1 and ACI 318 as follows:

- Structural steel minimum reinforcement ratio: $A_s / A_g \geq 0.01$
- Minimum longitudinal reinforcement ratio: $A_{sr} / A_g \geq 0.004$
- Maximum longitudinal reinforcement ratio: $A_{sr} / A_g \leq 0.08$
- Minimum number of longitudinal bars:
  ACI 318 Section 10.9.2 requires a minimum of four longitudinal bars within rectangular or circular members with ties and six bars for columns utilizing spiral ties. The intent for rectangular sections is to provide a minimum of one bar in each corner, so irregular geometries with multiple corners require additional longitudinal bars.
- Clear spacing between longitudinal bars:
  ACI 318 Section 7.6.3 requires a clear distance between bars of 1.5db or 38 mm (1.5in.).
- Clear spacing between longitudinal bars and the steel core:
  AISC Specification Section I2.1e requires a minimum clear spacing between the steel core and longitudinal reinforcement of 1.5 reinforcing bar diameters, but not less than 38 mm (1.5 in.).
- Concrete cover for longitudinal reinforcement:
  ACI 318 Section 7.7 provides concrete cover requirements for reinforcement. The cover requirements for column ties and primary reinforcement are the same, and the tie cover was previously determined to be acceptable, thus the longitudinal reinforcement cover is acceptable by inspection.

要注意的是，在AISC之I1.1 （1）里面为包含ACI318之10.13的复合截面柱的规定，因此在依照AISC之I的规定设计复合截面柱的时候，无需满足ACI 318 之10.13.8关于箍筋的规定。

当采用螺旋箍筋时，根据AISC I2.1a的最后使用说明中规定，要求满足ACI 318之7.10和10.9.3的要求。

附加箍筋尺寸的限制

ACI 318 之7.10.5.1要求所有的非预应力钢筋必须与侧向箍筋箍轧，对于小于32mm的纵向钢筋至少10mm（3号），对于36mm（11号）、43mm（14号）、57mm（18号）以及捆绑钢筋至少13mm（4号）。

最大的箍筋间距不能超过最小的柱子尺寸的一半

$$s_{max} = 0.5 \cdot \min \begin{Bmatrix} h_1 = 3072\text{mm} \\ h_2 = 3072\text{mm} \end{Bmatrix} = 1536\text{mm}$$

混凝土保护层

ACI 318之7.7章包含了混凝土的保护层厚度要求。对于不暴露于大气或埋在地下的混凝土，柱子混凝土的保护层厚度为38mm（1.5英寸）。

箍筋为纵向钢筋提供必需的侧向支撑

AISC之I2.1a的条纹说明中参用了ACI318 之第七章，附加横向箍筋的要求。按照ACI318之7.10.5.3的规定和图R7.10.5，要求支撑纵向钢筋的箍筋净间距大于150mm。角部的钢筋由周边箍筋支撑。

纵向钢筋及结构钢骨的配钢率的要求见AISC 之I1.1（4），I2.1 和ACI318， 如下：

- 钢骨最小配钢率：
- 纵向钢筋最小配筋率：$A_s / A_g \geq 0.01$
- 纵向钢筋最大配筋率：$A_{sr} / A_g \geq 0.004$
- 最小纵向钢筋数量：$A_{sr} / A_g \leq 0.08$
  ACI 318之10.9.2要求，使用有箍筋的矩形柱或圆柱最小要有4根纵筋；螺旋箍筋的柱子最少6跟纵筋。也就是说在巨型柱中至少每个角上都要有一根纵筋，几何不规则截面会有多个角，因此要求额外的纵筋。
- 纵筋净间距：
  ACI 318之7.6.3规定纵筋间净距1.5 纵筋直径或38mm。
- 纵筋与钢骨直接的净距：
  AISC 之I2.1e 规定最小的纵筋到钢骨的净距为1.5倍的纵筋直接但不能小于38mm。
- 钢筋的保护层厚度：
  ACI 318之7.7节给出了混凝土保护层厚度的要求。柱子箍筋和主筋的保护层厚度相同， 首先满足箍筋的保护层厚度，则纵筋的保护层厚度可他能通过验收测量。

数值方法检验M-N作用曲线

比利时Liège大学开发了有限元软件FinelG用以验证这一方法。软件运用材料法则比塑形方法更为准确。混凝土被定义为矩形曲线， 定义如下：

$$\sigma_c = f_c' \cdot \frac{\varepsilon_{cu}}{\varepsilon_c} \cdot \left(2 - \frac{\varepsilon_{cu}}{\varepsilon_c}\right)$$

$$\varepsilon_c = \frac{2 \cdot f_c'}{E}$$

其中，ε_{cu} = 0.003 - 无约束混凝土的极限受压变形（AISC I1.2b），ε_c - 强度达到峰值时的变形，E = 38007MPa。

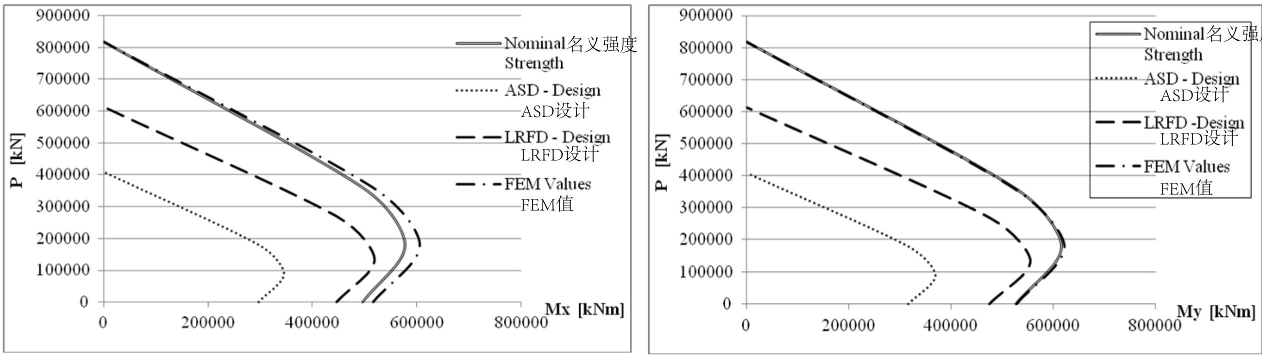


Figure 8. Comparison LRFD – ASD – Finite element analysis (Source: Plumiecs & ULg)
图8. LRFD与ASD比较——有限元分析 （出自： Plumiecs & ULg）

Numerical Methods for Checking M-N Interaction Diagram

In order to verify the method, the finite element software FinelG, developed at University of Liège (Belgium), has been used. The software uses material laws which are more accurate than in the plastic method. The concrete law is defined as a parabola rectangle diagram, and is defined as follows:

$$\sigma_c = f_c' \cdot \frac{\varepsilon_{cu}}{\varepsilon_c} \cdot \left(2 - \frac{\varepsilon_{cu}}{\varepsilon_c}\right)$$

$$\varepsilon_c = \frac{2 \cdot f_c'}{E}$$

Where: ε_{cu} = 0.003 – ultimate compressive strain of unconfined concrete (AISC I1.2b), ε_c – strain at reaching maximum strength, E = 38007MPa.

Tensile strengths are not taken into account in determination of the bending moment value. An elastic, perfectly plastic law is used to define the steel material, where: Fy = 450MPa, E = 200000MPa.

Comparison of the Results

The most interesting results for designers are the M-N interaction curves (see Figure 8).

The curve "nominal" is obtained by using the nominal strength values of material and a plastic block stress distribution. Design values of M-N interaction diagram have been obtained on the basis of a simple method presented in LRFD as well as in ASD. The results of the finite element study made with more refined models confirm the validity of the results obtained by that the simple method in the case of composite sections with several encased steel profiles (compare "nominal" and "FinelG" in Table above).

Conclusion

This paper describes an example of how to design a rectangular reinforced concrete column containing four encased steel profiles, untied between the floors. The calculations, based on the principles of AISC codes, lead to the design below, allowing an important reduction of the section dimensions for comparable section resistance in comparison to a normal reinforced concrete column, thanks to the use of high-strength heavy shapes (see Figure 9).

This is a general method which is suitable to be extended to more complex shapes with several encased jumbo sections in order to fit the requirements of the given project. Further documentation and the detailed calculation notes of this example are available upon request to the corresponding authors.

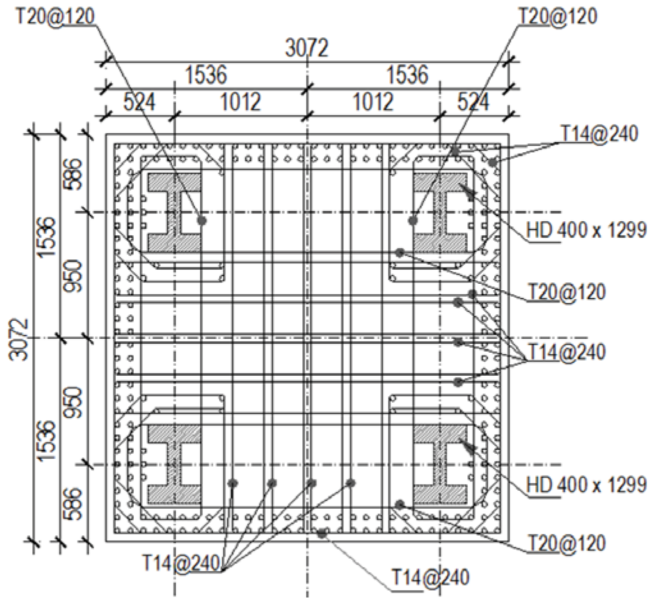


Figure 9. Final cross section using multiple Jumbos untied between the floors (Source: Plumiecs & ULg)
图9. 层间无联系的多个型钢加劲的最终截面图 （出自： Plumiecs & ULg）

在确定弯矩值时不考虑抗张强度。弹性、完全塑性法则用来定义钢材，其中Fy = 450Mpa，E = 200000Mpa。

结果比较

设计师比较感兴趣的结果是M-N作用曲线。（见图8）

这个曲线名义上是由材料的名义强度和塑形区域应力分部得到的。在LRFD和ASD中阐述了以简单的方法求得M-N作用曲线的设计值。用精细模型假以有限元方法研究的结果验证了由几个型钢加劲的复合截面用简单方法结算结果的有效性（“名义”计算结果和“FinelG”方法的对比见上表）。

结论

此文阐述了由4根层间无联系的型钢加劲的矩形混凝土柱的设计案例。计算基于AISC规定进行设计，在提供相同的承载力前提下，使得柱子的截面尺寸相对于普通的钢筋混凝土柱大为减小，这要归功于使用大截面高强度的H型钢。（见图9）

这是一种通用的设计方法可以引申至那些截面更为复杂的，需要有几个大型型钢钢骨的复合截面，以满足目前的一些复杂项目的要求。关于本文示例的更深入细致的论文和计算可以联系相关的作者索取。

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