

ATTEL PROJECT

PERFORMANCE-BASED APPROACHES FOR HIGH STRENGTH TUBULAR COLUMNS AND  
CONNECTIONS UNDER EARTHQUAKE AND FIRE LOADINGS

## Deliverable 2 – Design of the specimens

D2.1: Report on the design of specimens

D2.2: Definition of practical solutions for the selected typologies of column bases, of HSS-CHS columns and HSS-CFT columns and of HSS-concrete composite beam-to-column joints

### Contributing Partners:

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University of Trento

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# I Introduction

The objectives of WP2 are defined as follows:

- choice and optimization of the specimens to be tested;
- design and numerical modelling of HSS column and HSS-CFT column to be tested under earthquake and fire loadings;
- selection and design of HSS-concrete composite beam-to-column joints to be tested under earthquake and fire loadings;
- selection and design of base joints to be tested;
- estimation of specimens characteristics by means of F.E. models and calibration of models.

To define the specimens to be tested, it was agreed at the kick-off meeting held in Liège to extract these specimens from “actual” study cases in order to test realistic elements. Accordingly, it was proposed to design three different reference buildings (to be designed by the three Universities involved in the project (see Table 1)) corresponding to different loading conditions as described in Table 1. The objectives through the design of these reference buildings is i) as mentioned before to propose realistic configurations for the structural elements to be tested in laboratory (i.e. columns, beam-to-column joints and column bases) and ii) to show the benefit of the use of HSS, compared to the use of the “normal” steel.

The work accomplished in WP2 respected the plan of the activity as reported in the bar chart included in the contract; WP2 was finalised at the end of the first year. The definition of details and the final design of columns, beam-to-column joint and base-joints is finalised. The effectiveness of the proposed solutions will be demonstrated by the refined seismic and thermal analyses under execution within the other work packages. Moreover the drawing of the specimens to be tested in the laboratories have been produced in order to start with the fabrication of the specimens.

In the following paragraph, the properties of the designed structures are summarised. The details of the performed designs are given in the appendixes of the present document.

Table 1. Structural designs proposed for each university

	<b>ULG – Building I</b>	<b>UNITN – Building II</b>	<b>UNITH – Building III</b>
<b>Loading conditions</b>	Static + Fire	“Medium” earthquake (<0,25g) + Fire	“Strong” earthquake (>0,25g) + Fire
<b>Structural elements extracted from the designed buildings</b>	Beam-to-column joints	Beam-to-column joints	Beam-to-column joints
	Columns	Columns	Columns
	Column bases	Column bases	Column bases
	TESTED SPECIMENS SELECTED AMONGST THE DESIGNED STRUCTURAL ELEMENTS		

## II Design of Building Type 1

Before entering in the design of the building for such loading conditions, a preliminary economical study was performed to identify the situations when the use of HSS tubular columns is cost-effective (see Appendix 1).

The conducted study is a parametrical investigation performed on tubular columns. An optimization of the design has been performed on columns for different loading conditions (reflecting what can be met in practice in actual braced and unbraced buildings), different column heights and different steel grades (from S235 to S690); the obtained solutions for the different considered steel grades have been compared in term of cost.

Through this study, it has been demonstrated that the use of HSS columns is interesting for braced frames in which the columns support significant axial loads and low bending moments.

From the remarks of the economical study, a building configuration is proposed and designed. The design of a shopping centre with a braced composite structural system and HSS tubular columns was contemplated to define the reference building (see Appendix 2). The general layout of the designed building can be seen in the following figures.

The loads which were considered are the loads recommended in the Eurocodes for such a building.

The main beams and the secondary beams have been designed as composite ones as illustrated in Figure 3. For the design of the secondary beams, two spans have been contemplated: 8 m and 12 m (length B on Figure 1).

Three different column configurations have been considered as illustrated in Figure 4.

Through the design, different profiles for the beams and the columns were proposed for the different spans of secondary beams and the different column configurations (see Appendix 2). Through the performed investigations, it was illustrated that only the pure steel solution for the HSS columns can be economically interesting. So, for the definition of the specimens to be tested, only this configuration will be considered. In case of fire, it is recommended to protect these pure HSS columns appropriately (as it would have been the case with “normal” steel).

Also, for this building, configurations for the beam-to-column joints and the column bases are proposed as illustrated in Figure 5. These are the ones which will be experimentally tested within WP3 and WP4.

As the quantities of specimens to be tested are limited, taking into account the discussion in the design of reference building and considering practical conditions, the “static” specimens to be tested were defined; as a result, all the detailed drawings of the specimens were sent to Stahlbau Pichler for production.

For WP2, the performed work was dedicated to the justification of the chosen solutions, which will be tested within WP3 and WP4, and to their design. Within ATTEL project, this study will be completed in order to propose design solutions for buildings where the use of HSS tubular columns give an economical interest; for the column bases and the beam-to-column joints, the proposed

design models will be validated through comparisons to experimental tests (WP3 and WP4) and to numerical results (WP5) . The proposed design models will be part of WP6 results.

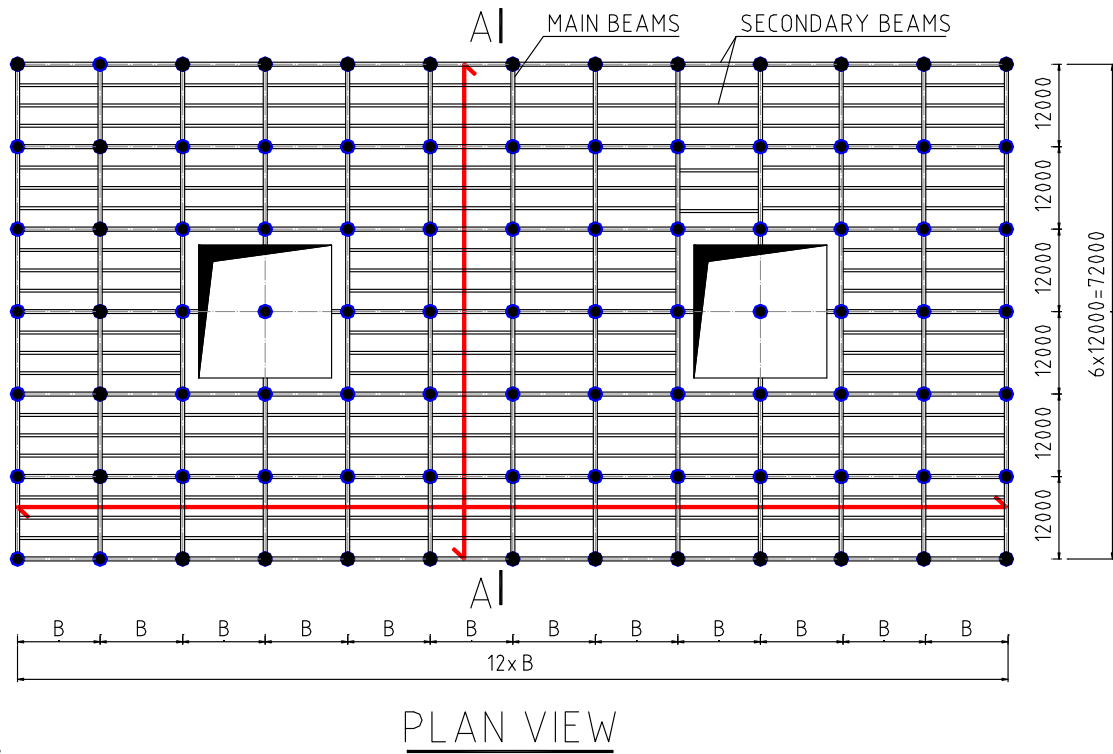


Figure 1. Plan view of the design building

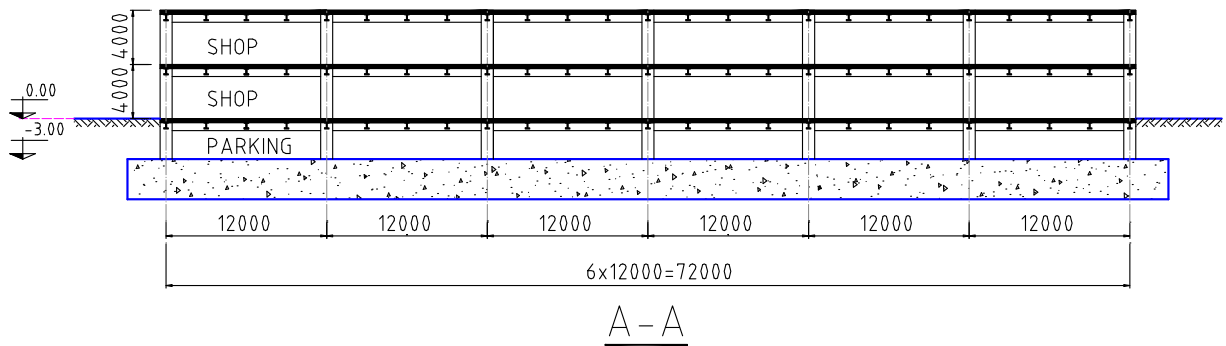


Figure 2. Main frame of the designed building

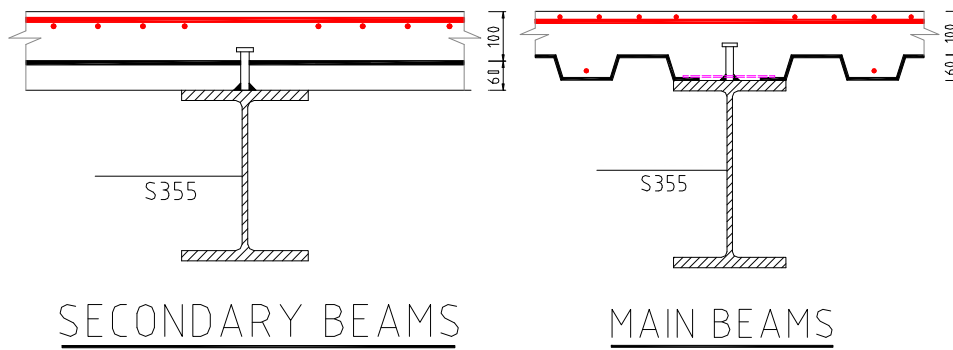


Figure 3. Designed composite beams

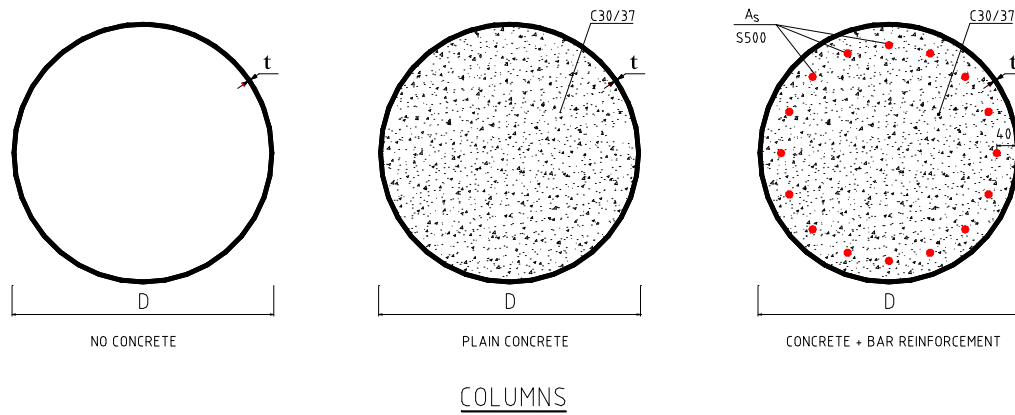


Figure 4. Contemplated column configurations

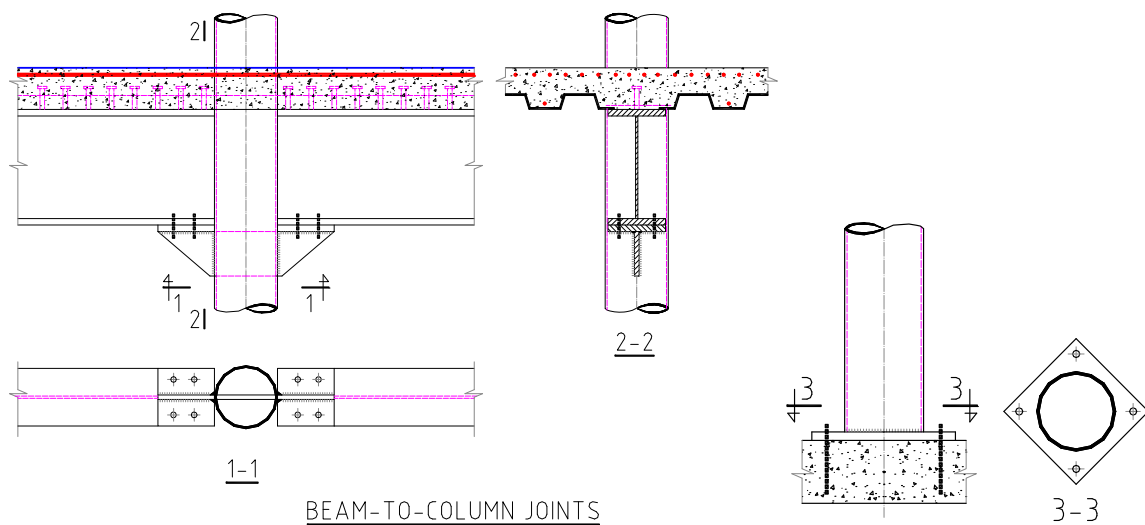


Figure 5. Proposed beam-to-column joint and base joint configurations to be tested

### III Design of Building Type 2

The proposed second solution of structure is a composite steel-concrete structure with tubular columns and composite beams with a composite concrete slab. In order to better satisfy design criteria for static, seismic and fire situations, a moment resisting frame along the main direction was designed, whereas concrete walls along the secondary direction are inserted, obtaining in this direction a pinned system. This choice permits to realize a well performing structure, because the seismic action for a moment resisting frame in both directions could be too severe for columns and joints design.

The structure is characterized by the following geometrical dimensions:

- Building category: office area (B);
- 5 storeys and inter-storey height equal to 3,5 m;
- Plan dimensions equal to 32 x 32 m, with square meshes 8 x 8 m;
- Frame structure in main direction Y (bending resistance moment);
- Pendulum frame in secondary direction (X).

- Open space in the first three storeys inside the building. Its plan dimension is 16 x 16 m.

This choice has been made in order to realize a bigger compartment and make an interesting study of the structural behaviour under fire and earthquake loads. Two staircases are localised between the concrete walls. These are necessary to evacuate building during fire or earthquake. In order to achieve the objective of the ATTEL project, the work focused on analysis and design of four types of columns:

- Circular hollow columns made with mild steel (*Normal Steel = NS*);
- Circular hollow columns made with high strength steel, (*High Strength Steel = HSS*);
- Mild composite columns CFT (*Composite Section with Normal Steel = CSNS*);
- High strength composite columns CFT (*Composite Section with High Strength Steel = (CSHSS)*).

The target consists to study and to compare performances of different tubular columns types under the seismic ( $a_g < 0.25g$ ) and fire actions, in order to investigate the actual possibility of using steel and steel-concrete composite HSS columns in order to satisfy the criteria of capacity design and fire resistance.

A static design of the building was first performed. Then, a verification of the building subjected to seismic loading was performed with the objective to determine if the use of HSS is useful both to respect capacity design criteria and to increase the column resistance under seismic loads. Through the performed studies, it was illustrated that the use of HSS columns satisfies the capacity design requirements given in Eurocode 8, without changing the diameter or the thickness of the column. Different is the case of the normal mild steel, where the designed steel or composite steel-concrete sections for the static design situation are not able to satisfy the seismic requirements or actions in the seismic design situation.

Also, configurations for beam-to-column joints and column bases are proposed as illustrated here below.

The beam connection to circular steel tubes presents more difficulties if compared to the I shape. Analytical results suggested that connections which transfer load from the girder to the concrete core potentially offer better seismic performance than connections to the steel tube alone. In fact, connections to the steel tube alone may exhibit large distortion of the tube wall around the connection region.

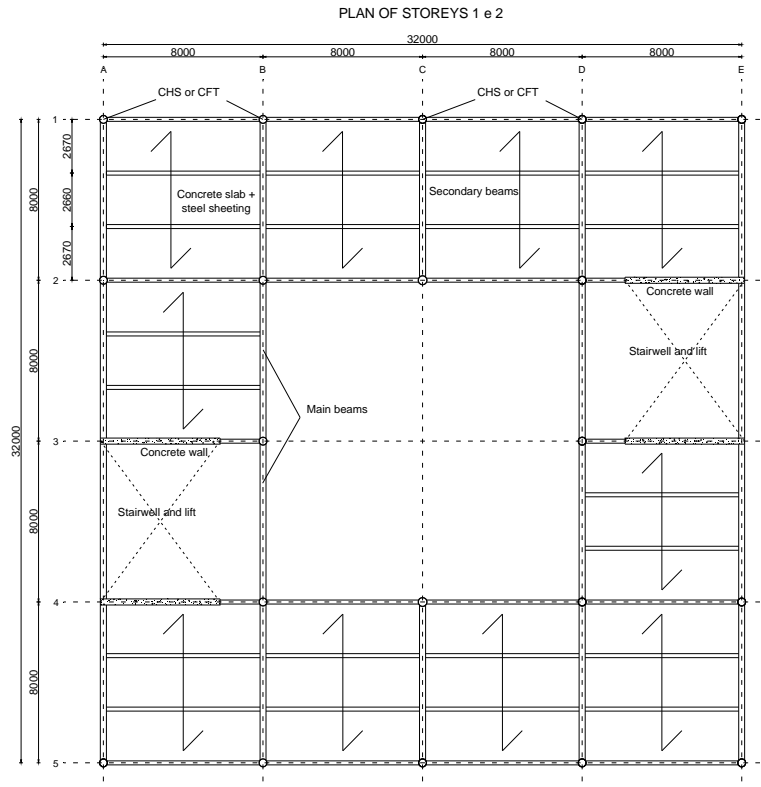


Figure 6. Plan view of a typical storey

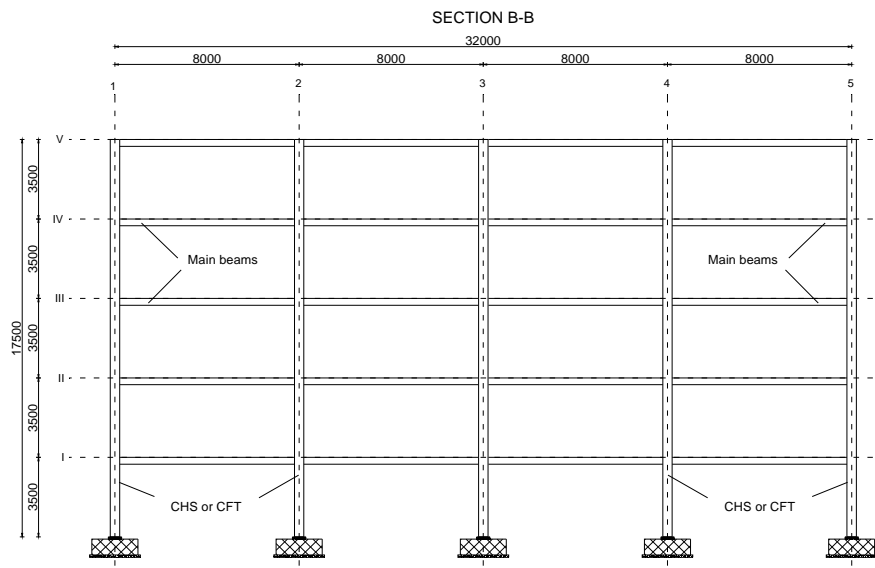


Figure 7. Building elevation view for section B-B

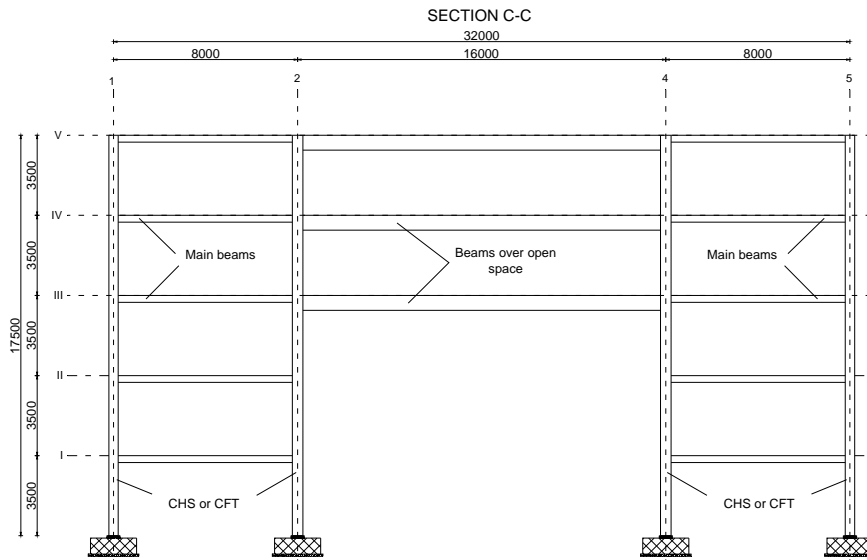


Figure 8. Building elevation view for section C-C

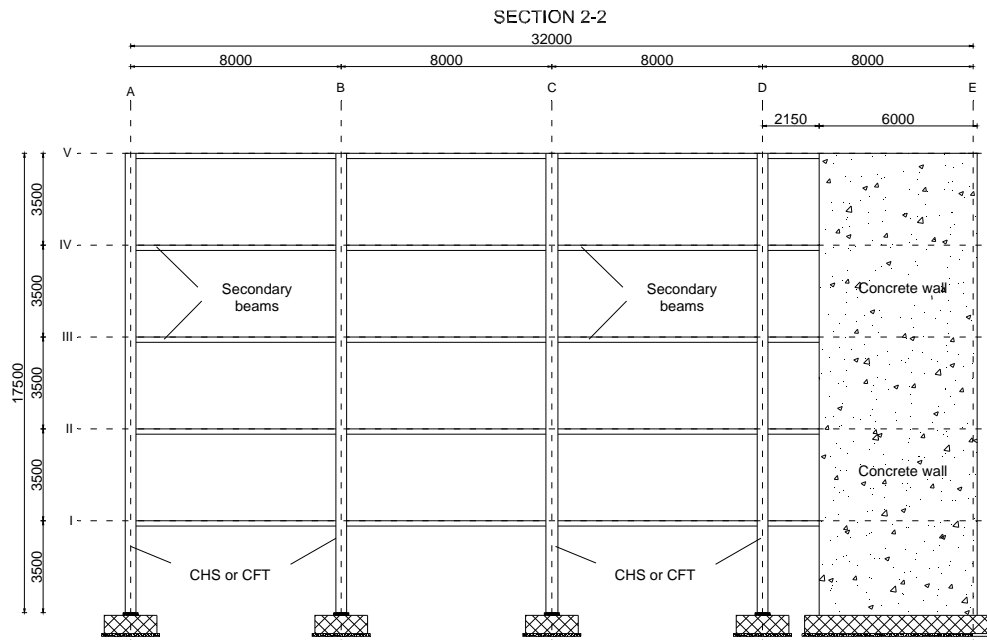


Figure 9. Building elevation view for section 2-2

Besides, components transferring girder forces into the concrete core exhibit better strength and stiffness characteristics than a simple connection to the tube face. However, the improvement in behaviour depends also on the type of component penetrating the concrete core. Moreover, the choice to obtain a collapse mechanism involving the formation of plastic hinges at the beams' ends required the realization of a beam-to-column joint possessing enough overstrength with respect to adjacent beams. Besides, by taking into account the difficulties related to the erection of this joint typology, it was decided to realize a rigid full strength connection.

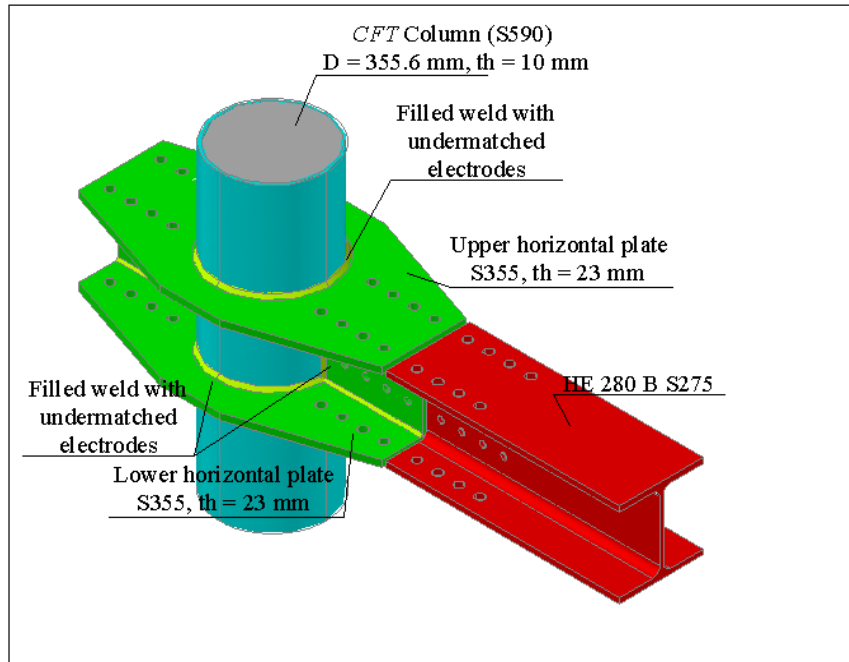


Figure 10. View of the beam-to-column moment resisting joint

The proposed welded/bolted solution was conceived to guarantee easiness of assembly and limited problems related to on site welding. The joint was made by two horizontal diaphragm plates and a vertical through-column plate attached on the pipe by groove welds. Flanges and web of each beam were connected to the horizontal plates and the vertical plate respectively by cover joint plates and two and three rows of bolts M27 and M20 8.8 as illustrated in Figure 11.

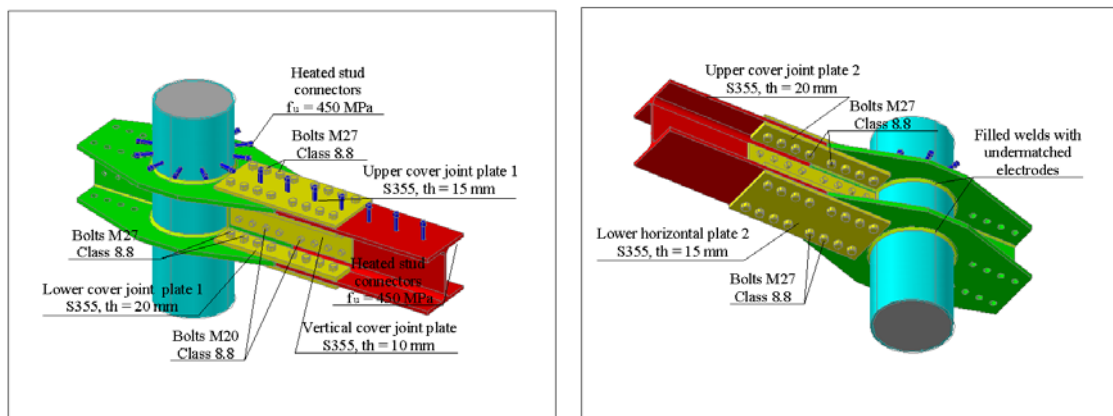


Figure 11. View of the beam-to-column moment resisting joint

For the column bases, two different solutions are proposed:

1. Standard solution with a base plate, anchor bolts and vertical stiffeners:

The base plate is welded around the perimeter of the column, the stiffeners are welded on column and base plate, the anchor bolts are inside the concrete foundation.

2. Advanced solution with column embedded in the foundation:

The idea is to satisfy the capacity design criteria by embedding the column inside the foundation. This solution is surely more performing for CFT columns. In other European projects (O. S. Bursi, S. Caramelli, G. Fabbrocino, J. Molina, W. Salvatore, F. Taucer and R. Zandonini “3D Full-Scale Seismic Testing of a Steel-ConcreteComposite Building at ELSA”, EUR 21299 EN, 2004), it was noted that the first joint can result semi-rigid under cyclic loads. The reason of this behaviour is the elongation of the anchor bolts in tension when the grout will be damaged owing to the cyclic action of the earthquake. For this reason this second solution is proposed because it should perform better as rigid and full-strength base-joint.

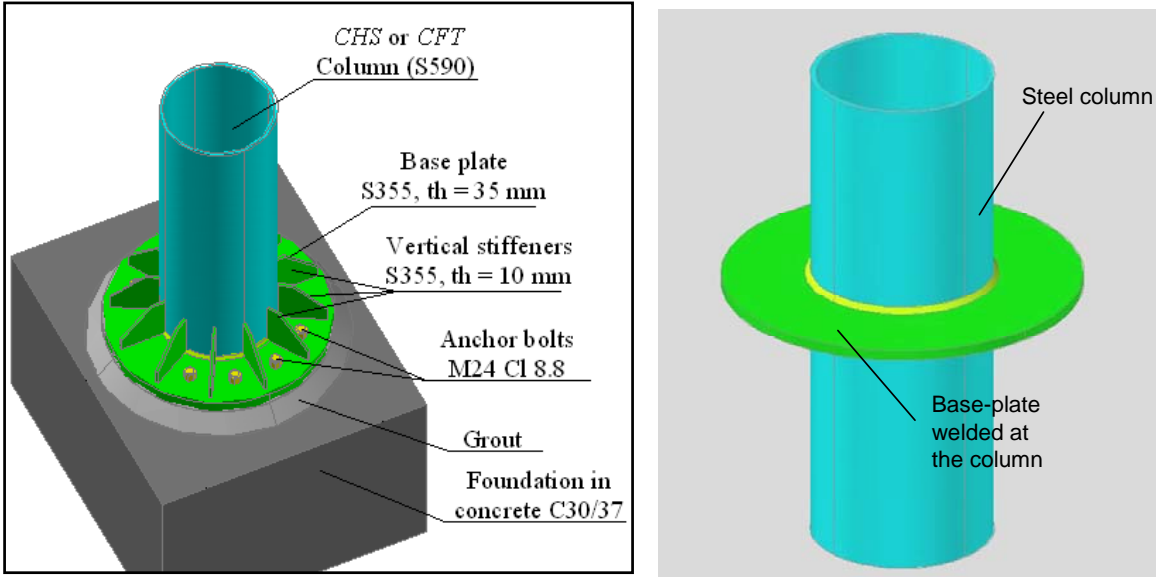


Figure 12. Base joint: Solution 1 and Solution 2

The proposed innovative base-joint with embedded column is designed according to the Strut&Tie mechanism for prefabricated concrete construction (pt. 10.9.6 EN 1992-1-1:2005) considering that the forces coming from the column in term of axial load, shear load and moment action are carried out by three forces forming in the foundation, according to the equilibrium system reported below.

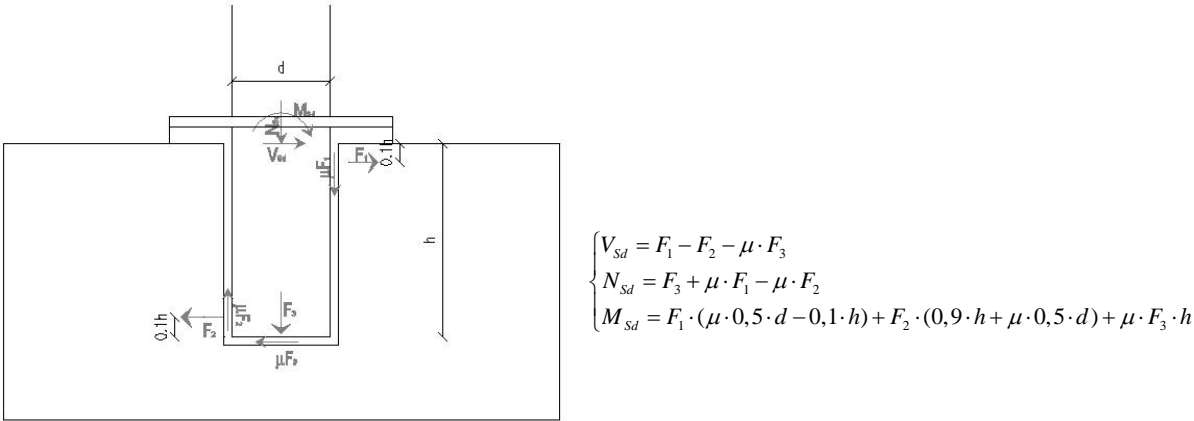


Figure 13. Base joint: Solution 1 and Solution 2

These three forces ( $F_1$ ,  $F_2$  and  $F_3$ ) are transferred in the foundation by the formation of concrete struts (frontal strut and diagonal struts) equilibrated by tension tie supported in the rebars.

According to this hypothesis, some formulas are then proposed for the characterisation of the resistance of the concrete strut and the design of the rebars that we need to add in the foundations.

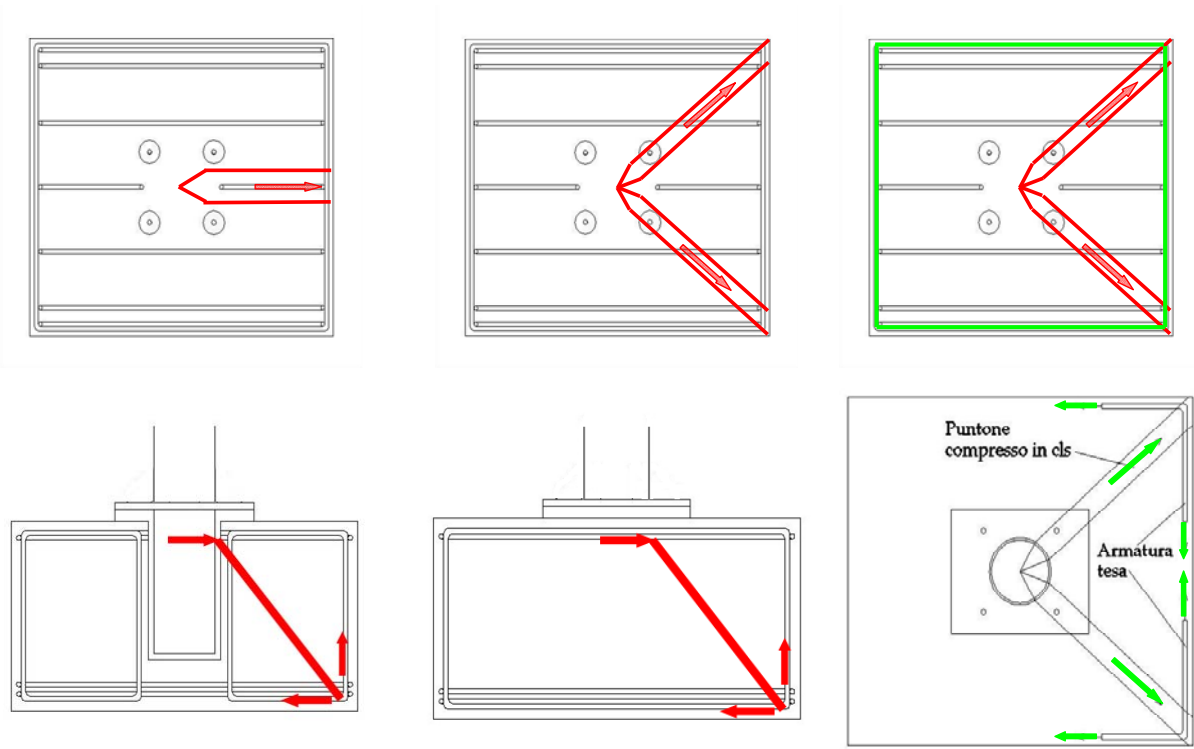


Figure 14. Strut&Tie transfer mechanism in the foundation

For the fire situation, the over-design criteria is to exploit the over-resistance offered by the HSS to reach, under fire loads, better performance without protection or increased size, in other words without changing the sectional factor of the structural element.

The four types of columns previously designed under static loads are subjected to fire action with the help of the finite element program for non-linear analysis, SAFIR, developed by University of Liège (J-M. Franssen, “SAFIR: A thermal/structural program for modelling structures under fire”, Engineering Journal 2005, 143-158). According to the design procedure contained in EC1-1-2, 2D numerical model was performed considering different fire scenario (fire in the small compartment, fire in the open-space, fire in the full ground floor) and the implementing of the ISO curve (perspective approach) and natural fire curve (performance-based approach) depending on: the design fire load; the presence of active prevention system; the dimension of the compartment; the dimension and location of the safety exit doors; the components of the partitions. Through the performed investigations, it was demonstrated that it is possible to reach the fire resistance requirements imposed by Eurocodes and National Standards (R60) by the use of HSS CHS columns without protection. Moreover, it is possible to reach exposure time greater than R60 with CFT columns and with or without HSS sections. The results are summarized in the following Table.

The detailed procedure adopted for the design of the structure and the relevant joint configurations (beam-to-column joint and base-joint) are described in Appendix 3 given at the end of this document.

Table 2: Fire resistance for the different structural solutions

Prototype structure with CHS columns 406 x 16 – 406 x 12					
	Prescriptive Approach R [min]			Performance-based app R [min]	
Steel Grade	S 355	S 590 Q	S 590 Q <sup>(*)</sup>	S 355	S 590 Q
Fire in all ground floor	28	34	31	38	43
Fire in Open Space	55	<b>65</b>	<b>65</b>	55	<b>65</b>
Fire in lateral	53	<b>63</b>	59	53	59

(\*) HSS with reduction factor proposed in the literature

Prototype structure with CFT columns 355 x 12 – 355 x 10 - C30/37 - 8 ϕ 18					
	Prescriptive Approach R [min]			Performance-based app R [min]	
Steel Grade	S 355	S 590 Q	S 590 Q <sup>(*)</sup>	S 355	S 590 Q
Fire in all ground floor	<b>79</b>	<b>79</b>	<b>80</b>	<b>91</b>	<b>91</b>
Fire in Open Space	<b>100</b>	<b>100</b>	<b>100</b>	<b>100</b>	<b>100</b>
Fire in lateral	<b>91</b>	<b>91</b>	<b>90</b>	<b>92</b>	<b>90</b>

(\*) HSS with reduction factor proposed in the literature

## IV Design of Building Type 3

On the basis of the results obtained from the previous study case it evidenced that, for tall moment resisting frames (with four or five stories) under high seismic actions, the design is governed by the satisfaction of the damage limit states instead of the ultimate limit states. This means that for this type of structures, in very high seismic regions, the use of the HSS is not useful being important the dimension of the columns in order to satisfy the limitation of the interstorey drift for the DLS and the interstorey drift sensitivity coefficient in order to consider the second order effects.

For this reason, in order to better identify a possible structural solution for the high seismic region, a five storey-composite building with braced and moment resisting framing system was proposed to be designed with high-strength steel CHS tubular columns and regular-strength steel I-beams with composite structural behaviour. This configuration corresponds to a typical office building, used in Greece.

The aim of this study is to compare performances of different tubular columns types mainly under strong seismic actions ( $a_g > 0.36g$ ). The main interest is to investigate the actual possibility of using steel and steel-concrete composite HSS columns that will satisfy all the Eurocode imposed criteria and will be competitive in terms of cost at the same time.

For the design of this reference structure all the prescribed loading actions and the corresponding loading combinations will be considered. The earthquake loading is the major design parameter and it is expected to govern the total design of the building for some of the selected structural typologies.

Following standard design practice in Greece, fire loading will be considered through appropriate paint or fiber-coating protection of the steel elements.

A preliminary analysis and design has been already conducted, accounting for the EN 1993 and EN 1998 provisions for strength and serviceability. For this preliminary design, CHS columns were considered with HSS (590 MPa) and I-beams from regular steel (355 MPa). The plan views and the description of the reference structure are illustrated in the following paragraphs.

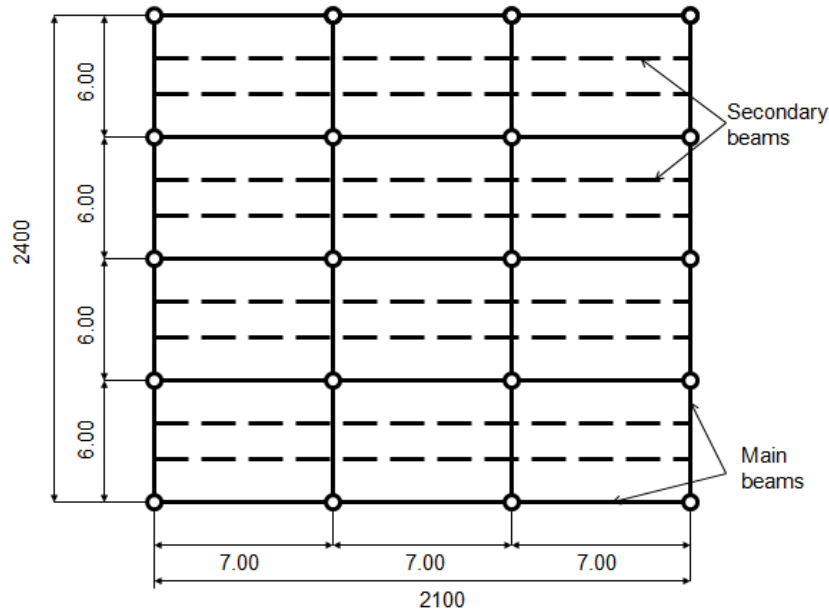


Figure 15. Plan view of the office building considered for design and analysis.

#### *IV.1.1.1 Moment resisting frame building*

The selected steel moment resisting frame (Figure 16) is a structural system in which beam-column connections are realized as full strength rigid joints. As a result, the frame members resist lateral loads through their flexural stiffness and strength. This structural system provides ductility to the structure and high energy absorption, making it ideal for seismic prone areas where the design is oriented towards this target. The drawback of this approach is the increased lateral deformation of the structure which has to be within the serviceability limits imposed by the Eurocode.

Moment resisting frames are able to provide an advanced energy dissipation mechanism, thanks to their large number of dissipative zones. Through this mechanism, the structure is expected to satisfy all the code imposed requirements. On the other hand, fulfilment of the requirements necessary to guarantee the serviceability limit state check becomes more and more difficult as the height of the building increases. This is due to the decreased lateral stiffness the framed structures possess, which causes high sway deflections even under moderate earthquake or wind actions.

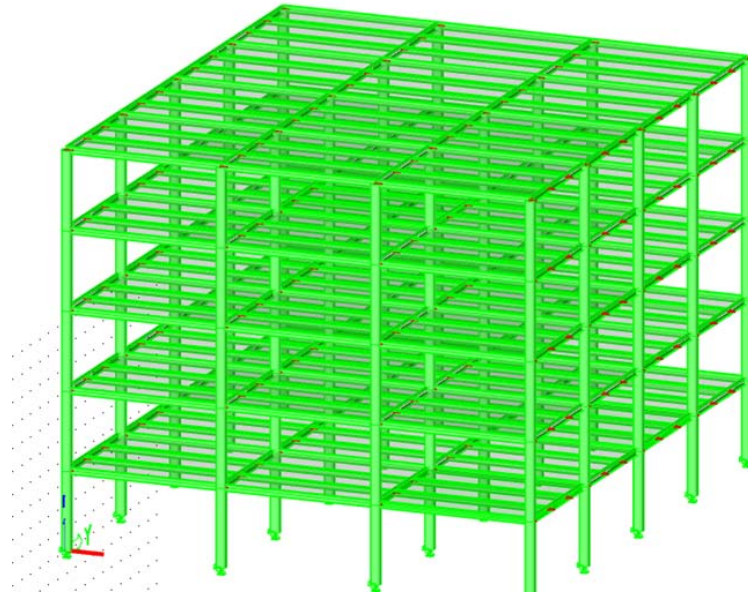


Figure 16. 3D elevation view of the moment resisting building.

#### *IV.1.1.2 Braced frame building “Type 1”*

Steel moment-resisting frames are susceptible to large lateral displacements during severe earthquake ground motions. Therefore, special attention is required in order to limit the damage to non-structural elements as well as to avoid problems associated with P- $\Delta$  effects and brittle or ductile fracture of beam to column connections [FEMA, 2000]. As a consequence, engineers in the US have increasingly turned to concentrically braced steel frames as an economical way to resist earthquake loads.

Concentrically braced frames can be designed to carry the total value of the seismic horizontal forces. According to Eurocode 8 and ECCS recommendations, the design value of the q-factor of X-braced frames is assumed equal to  $4a_u/a_v$ .

The X-bracing system consists of two diagonal braces pin-connected to a surrounding frame. The diagonal elements are subjected to compression and tension forces respectively, which cause increased axial load to the supporting columns. This has to be taken in to account for the column design.

The “**Type 1**” office building with concentric braces selected for design by UNITH is shown in the next figures.

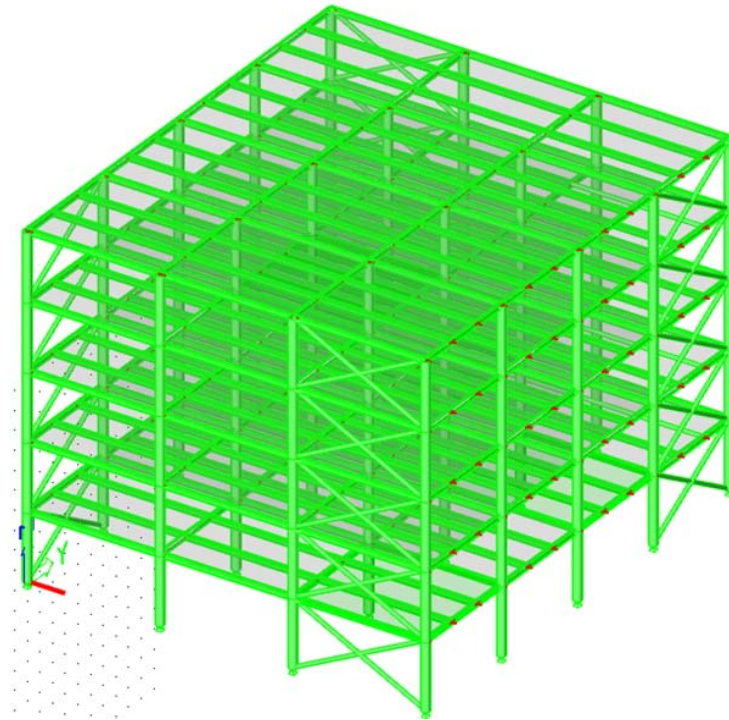
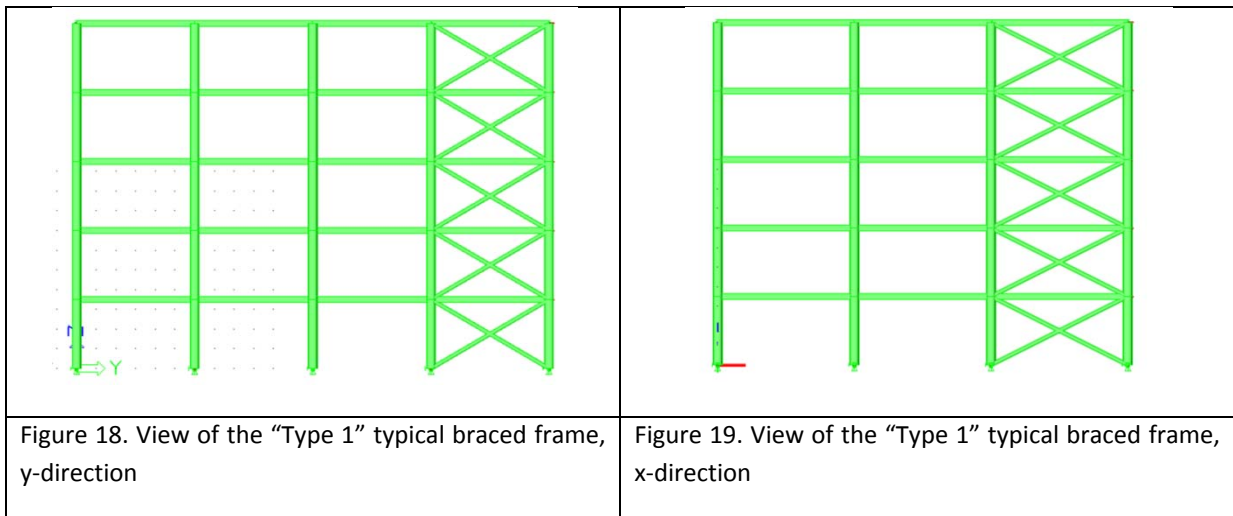


Figure 17. 3D view of the “Type 1” braced frame building



#### IV.1.1.3 Braced frame building “Type 2”

The proposed “**Type 2**” office building selected for design by UNITH is illustrated below.

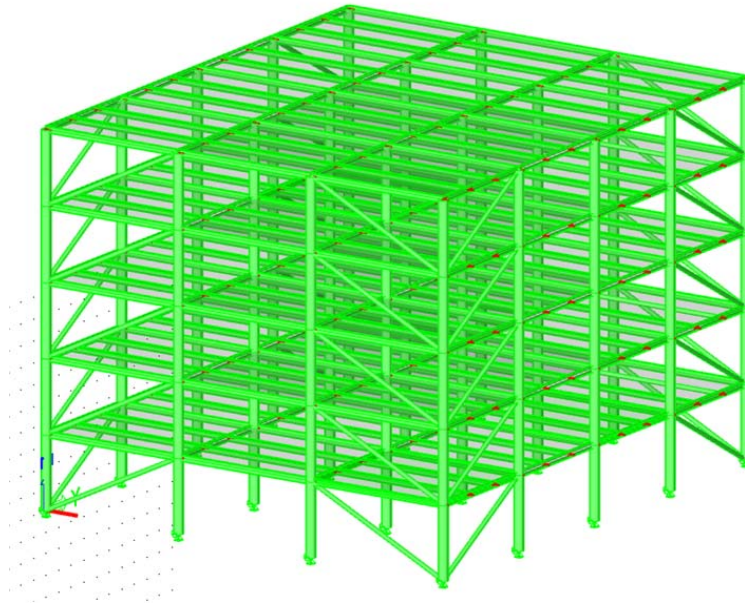
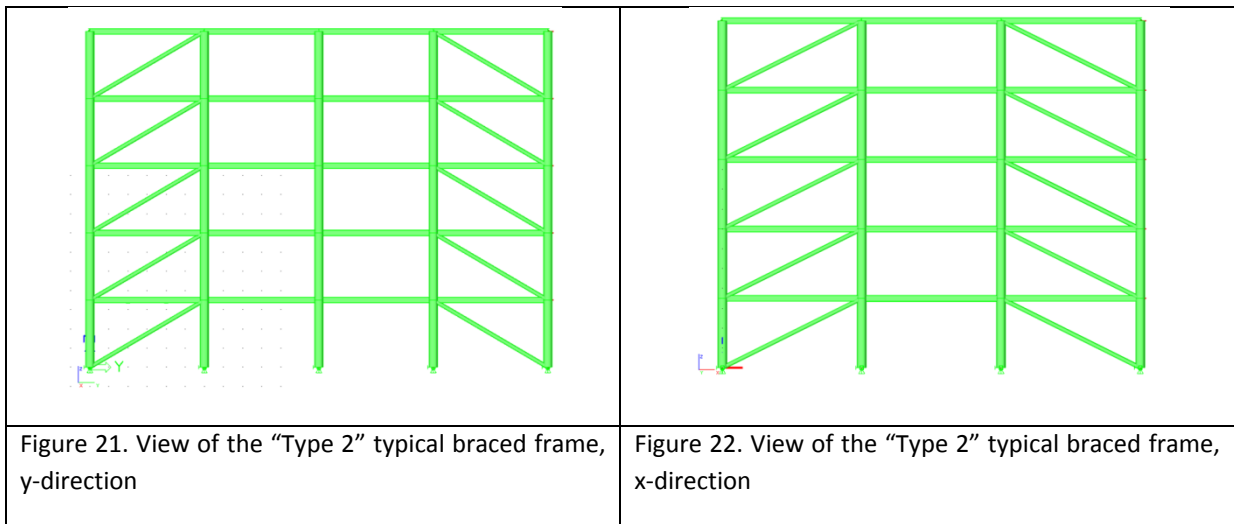


Figure 20. 3D view of the “Type 2” braced frame building



Preliminary 3-dimensional drawings for the moment resisting frame and braced frame connections are proposed as illustrated below. Moment resisting type of joint is decided to be rigid full strength connections in order to activate the formation of plastic hinges at the beams' ends as the beam-to-column joint should possess enough overstrength with respect to adjacent beams. The type of joint referring to the braced system is considered to be a simple transverse loading transfer connection.

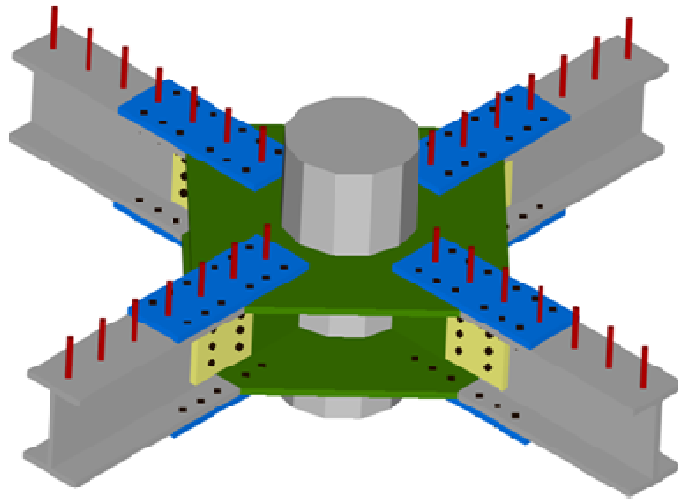


Figure 23. View of the beam-to-column moment resisting, rigid full-strength joint.

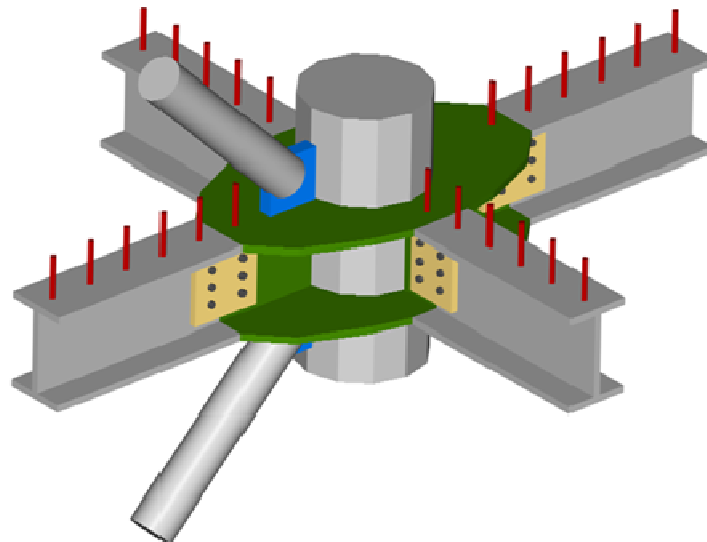


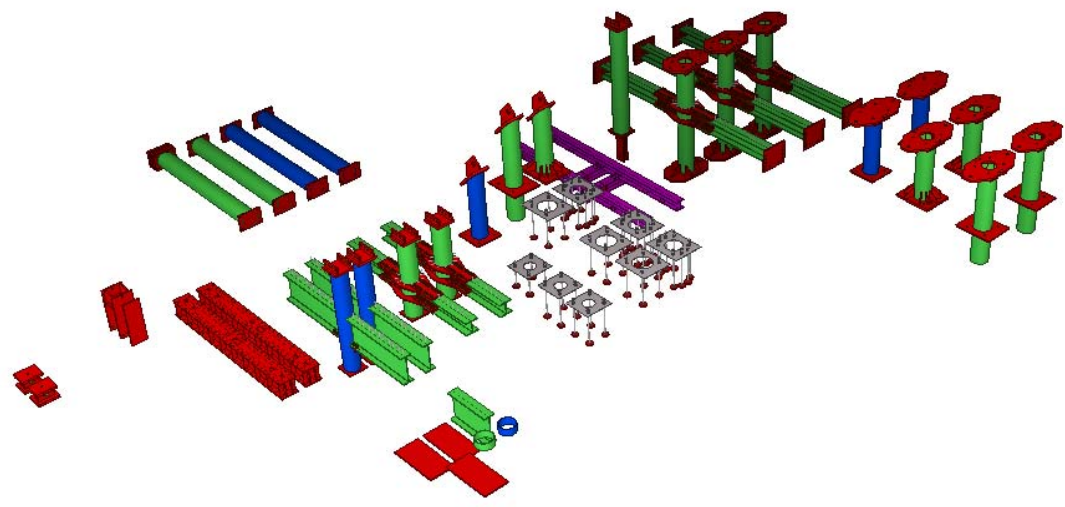
Figure 24. View of the beam-to-column braced-frame joint.

## V Fabrication of the specimens

From the previously designed structures, the following structural elements to be tested were extracted:

- Two different tubular HSS columns (S590 CHS 355x12 and S590 CHS 324x10);
- Two different types of beam-to-column joints (see Figure 5 and Figure 11);
- Three different types of column bases (see Figure 5 and Figure 12).

The drawings of the specimens to be produced have been prepared by STBPI, in collaboration with the different laboratories, as illustrated in the following figure produced by the program “TEKLA Structures”.



For each specimen, all the pieces for the fabrication are reproduced in singular drawings and then checked by each partner according to the requirements of the test set-up. Some examples of these drawings are reported here below; all the drawings are reported in Appendix 4.

All the specimens are produced with S275 and S355 steel grades for the beams and the plates and HSS 590 steel grade for the circular columns. For all the specimens, weld material undermatching the mechanical properties of HSS (S590) and overmatching the mechanical characteristic of the mild steel (S275 or S355) was selected. In detail a weld metal G 46 4 M G4Si1 is used.

Here below, some photos of the already produced specimens are reported. All the specimens should be delivered before the end of April.

**5 SPECIMENS**

**A - A**

Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso	
e1	PL20*85	S355JR	12	140	0.0	1.1	
e3	PL40*534	S355JR	2	534	0.4	47.1	
e31	RR355*12	T5590	1	1770	1.9	179.1	
Totale:						3.0	291.1

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Autografo autorizzato	tecnicista	disegnazione
Zuständige Berechnungen für Mista ohne Toleranzzugabe / Schweißlinie:		
Sostamenti ammessi per misure senza tolleranze / saldature:		
Standard Stahlbau Pichler		
Ciente: UNIVERSITA' TRENTO	STAHBAU PICHLER	
Objetto: ATTEL	CSM10	
Comessa: 1559	Data: 11.11.2009	Disegn: [Logo]
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CRE/DIS:	Geprüft / Controllato:	Datum/Date:
TR: Zeliger	ST: Sommerh	





# **Appendix 1: Economical study**

**UNIVERSITY OF LIEGE**

**Department ArGEnCO**

**MS<sup>2</sup>F Division**

**Field of application of high strength steel circular tubes for  
steel and composite columns from an economic point of  
view**

***ATTEL Project, Internal report***

Hoang Van Long

Ly Dong Phuong Lam

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Barbara Rossi

Jean-Pierre Jaspart

February 2009

## 1. Introduction

High strength steel is the term generally employed for steel presenting a proof strength higher than 450MPa. The use of high strength steel (HSS) in load-bearing structures has fairly during the last decades thanks to its advantages as underlined by authors in [1, 6, 7, 9]. The reason lays not only but mainly in its economic interest compared to steel, whose cost increases slower than its strength. However, as the stiffness of HSS structures is smaller than the one of normal steel (NS) structures, the second-order effects and the serviceability requirements considerably limit the use of HSS in load-bearing structures.

But this question has been sporadically considered in the researches concerning the behaviour of structures made of HSS. Therefore, the economic profit of using HSS in constructions needs to be more adequately studied in order to highlight the advantages of HSS in each type of structures. The present work aims at investigating steel and composite construction using circular steel tubes for the columns. The objective is to define the two respective domains where HSS and NS respectively are of economic interest. Two points will be simultaneously reported: (1) provide a general view of the economic benefit of the use of HSS; (2) establish the basis for choosing the material (HSS or NS) for framed structures before designing it.

The present research compares the costs of two columns made of HSS and NS. Steel with yield strengths varying from 500 N/mm<sup>2</sup> to 700 N/mm<sup>2</sup> will be considered as HSS [4] while S355 steel is considered as NS. The strength, stability and stiffness conditions according to Part 1-1 of Eurocode 3 [3] and Part 1-1 of Eurocode 4 [5] will be taken into account in the optimum cost design for steel and composite columns. Concerning the analysis of structures made of HSS, the rules of Part 1-12 of Eurocode 3 will be used [4]. Simple columns, columns in braced/un-braced frames and general frames will be investigated. In each case, the algorithms are implemented and the resulting automatic calculation allows examining almost practical possibilities.

## 2. Investigation for simple columns

Let us consider a simple column as the one depicted in Fig.1. The cost of two solutions with the same length and under the same loading but using two different steel grades, NS and HSS, is evaluated. In order to be comparable, the *optimum cost design* for each column is needed (Section 2.1). Afterwards, a global comparison is made in order to evaluate which grade is of economic interest (Section 2.2). An adequate number of case studies to be carried out is then chosen in order that the global study provides general results (Section 2.3). The conclusions for simple columns are provided in Section 2.4.

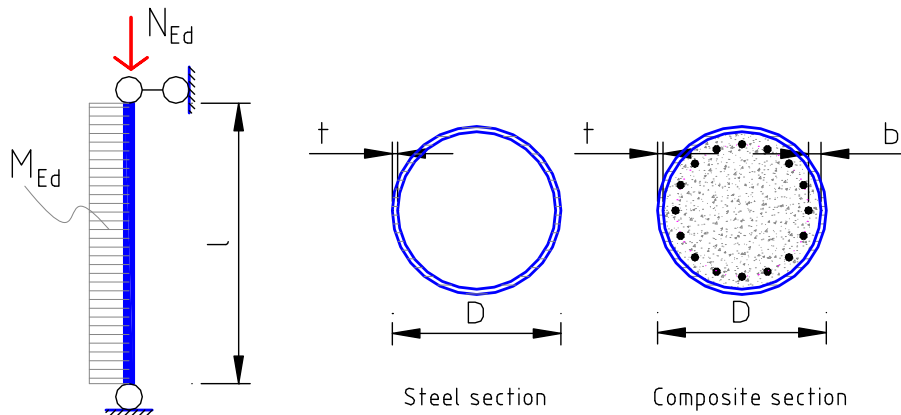


Fig.1. Single column

## 2.1. Optimum cost design for single columns

The *optimum cost design* provides the cheapest structure among the admissible solutions respecting the necessary safety conditions. Generally, an *optimum cost design* consists of three main steps: (1) the application of the rules controlling the safety of the structure that will be the constraints of the optimal problem (Sections 2.1.1 and 2.1.2); (2) the establishment of the cost function representing the objective to be minimized (Section 2.1.3); (3) the arrangement of the problem under mathematical optimal procedure, and the choice of a suitable algorithm to solve the problem (Section 2.1.3).

### 2.1.1. Safety condition for steel columns

The very widespread safety rules of Eurocode-3 [3] are used in the present work. Only a few recalls necessary for the good comprehension of the reader are provided below.

- The input quantities are:

- $f_y$  is the characteristic (k)/design (d) value of the yield strength;
- $E$  is the modulus of elasticity of the steel;  $E = 2,1 \times 10^9$  kN/m<sup>2</sup> is taken in the present work;
- $D$  is the outside diameter of the tube;
- $t$  is the tube thickness;
- $l$  is the length of the column, it is also the buckling length in this case.
- $N_{Ed}$ ,  $M_{Ed}$  are the respective design values of the compression and the bending moment.

If the bending moment is not uniform, the equivalent moment is used.

-Classification of sections:

$$\left. \begin{array}{l} D/t \leq 50\varepsilon^2 : \text{ Class 1;} \\ D/t \leq 70\varepsilon^2 : \text{ Class 2;} \\ D/t \leq 90\varepsilon^2 : \text{ Class 3;} \end{array} \right\} \quad (1)$$

where  $\varepsilon = \sqrt{235/f_y}$  if  $f_y$  is expressed in N/mm<sup>2</sup>.

Class 4 is not considered in the present work.

-The intermediate quantities:

- $A = \pi(t^2 + dt)$  is the cross-section area;
- $W_{el} = \frac{\pi}{32d}[d^4 - (d - 2t)^4]$  is the elastic section modulus;
- $W_{pl} = \frac{1}{6}[d^3 - (d - 2t)^3]$  is the plastic section modulus;
- $I = \frac{\pi}{64}[d^4 - (d - 2t)^4]$  is the second moment of inertia of the cross-section;
- $N_p = f_y A$  is the design plastic compression resistance;
- $M_{el} = f_y W_{el}$  is the design elastic bending resistance;
- $M_{pl} = f_y W_{pl}$  is the design plastic bending resistance;
- $i = \sqrt{\frac{I}{A}}$  is the radius of gyration of the cross-section;
- $\bar{\lambda} = \sqrt{\frac{N_{pl}}{N_{cr}}} = \frac{l}{\pi i} \sqrt{\frac{f_y}{E}}$  is the non-dimensional slenderness of the column where the elastic buckling force is provided in Eq. (2);

$$N_{cr} = \frac{\pi^2 EI}{l^2} \quad (2)$$

-  $\Phi = 0,5[1 + \alpha(\bar{\lambda} - 0,2) + \bar{\lambda}^2]$  is used to determine the reduction factor  $\chi$ , with:  $\alpha = 0,21$  for S355 and  $\alpha = 0,13$  for HSS, the tubes are supposed to be hot finished. The reduction factor is calculated using Eq. (3);

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \quad (3)$$

-  $C_{yy}$  is the factor taking into account the reduction of the design plastic resistance due to axial force:

$$C_{yy} = 1 \quad \text{for section of Class 3;}$$

$$C_{yy} = \begin{cases} 1,1 & \text{if } \bar{\lambda} \leq 1 \\ 1/[0,9 + 0,5(W_{pl}/W_{el} - 0,9)(\bar{\lambda} - 1)] & \text{if } 1 < \bar{\lambda} \leq 3 \\ (W_{pl}/W_{el})^{-1} & \text{if } \lambda > 3 \end{cases} \quad \text{for section of Class 1 or 2.}$$

-  $M_{Rk}$  is the characteristic bending resistance of the critical cross-section: with sections of Class 1 and Class 2:  $M_{Rk} = M_{pl}$ ; with sections of Class 3:  $M_{Rk} = M_{el}$ .

- Member verification:

$$\frac{N_{Ed}}{\chi N_p} + \frac{1}{1 - \frac{\chi N_{Ed}}{N_{cr}}} \frac{M_{Ed}}{C_{yy} M_{Rk}} \leq 1 \quad (4)$$

### 2.1.2. Safety condition for composite columns

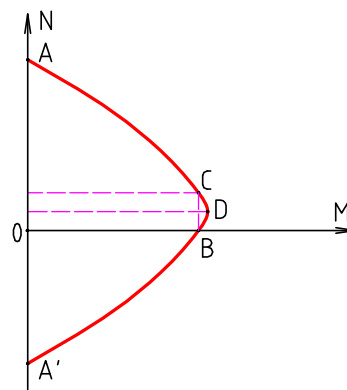
The rules of Eurocode-4 [5] for composite columns are summarized in this Section.

-The input quantities:

- $f_y$  is the characteristic (k)/design (d) value of the yield strength of the steel tube;
- $f_{ck}$ ,  $f_{cd}$  are the characteristic value and the design value of the strength of the concrete;
- $f_{sk}$ ,  $f_{sd}$  are the characteristic value and the design value of the strength of the rebar;
- $E_a$ ,  $E_c$ ,  $E_s$  are the modulus of elasticity of the steel, the concrete and the rebar. The values of  $E_c = 3,2 \times 10^8$  kN/m<sup>2</sup> and  $E_s = 2,1 \times 10^9$  kN/m<sup>2</sup> are used in the present work.
- D is the outside diameter of steel tube;
- t is the tube thickness ;
- b is the distance from the centre of the rebar to the inside face of the tube (Fig.1);
- $l$  is the length of the column, it is also the effective length in this case.
- $N_{Ed}$ ,  $M_{Ed}$  are the design values of the axial load and of the bending moment.

- Strength of the cross-section (Interaction curve)

Due to the non-symmetry of the stress-strain response of concrete under tension and compression, the shape of the interaction curve for composite sections is similar to the curve shown in Fig.2, with the particular points: A, B, C and D. With circular sections, the volume of computation to determine this curve by hand is quite large. Thus, in order to be able to implement the equations for automatic calculations, the following assumption is made (Figs. 3a and 3b): the area of reinforcements is supposed to be a continuous ring instead of distinct rebars. The ensuing error decreases when the number of rebars increases.



**Fig.2:** Interaction curve for circular composite section

Using the symbols reported in Fig.3, all the following quantities can be expressed using the location of the neutral axis that only depends on the angle  $\theta_1$  (Figs.3c and 3d):

$$r_3 = (r_2 - b) + \frac{A_s}{4\pi(r_2 - b)};$$

$$r_4 = (r_2 - b) - \frac{A_s}{4\pi(r_2 - b)};$$

$$\theta_i = \begin{cases} \arctan \frac{r_i \sin \theta_1}{\sqrt{r_i^2 - (r_i \sin \theta_1)^2}} & \text{if } r_i^2 - (r_i \sin \theta_1)^2 > 0 \\ \pi/2 & \text{if } r_i^2 - (r_i \sin \theta_1)^2 \leq 0 \end{cases};$$

$$A_i = r_i^2 (\pi/2 - \theta_i - \sin \theta_i \cos \theta_i);$$

$$W_i = \frac{2}{3} r_i^3 \cos^3 \theta_i;$$

$$N_{\theta_1} = (A_a + 2A_2 - 2A_1) f_{yd} + (A_c + A_3 - A_2 - A_4) f_{cd} + (A_s + 2A_4 - 2A_3) f_{sd}; \quad (5)$$

$$M_{\theta_1} = (S_1 - S_2) f_{yd} + (S_2 - S_3 + S_4) f_{cd} + (S_3 - S_4) f_{sd}. \quad (6)$$

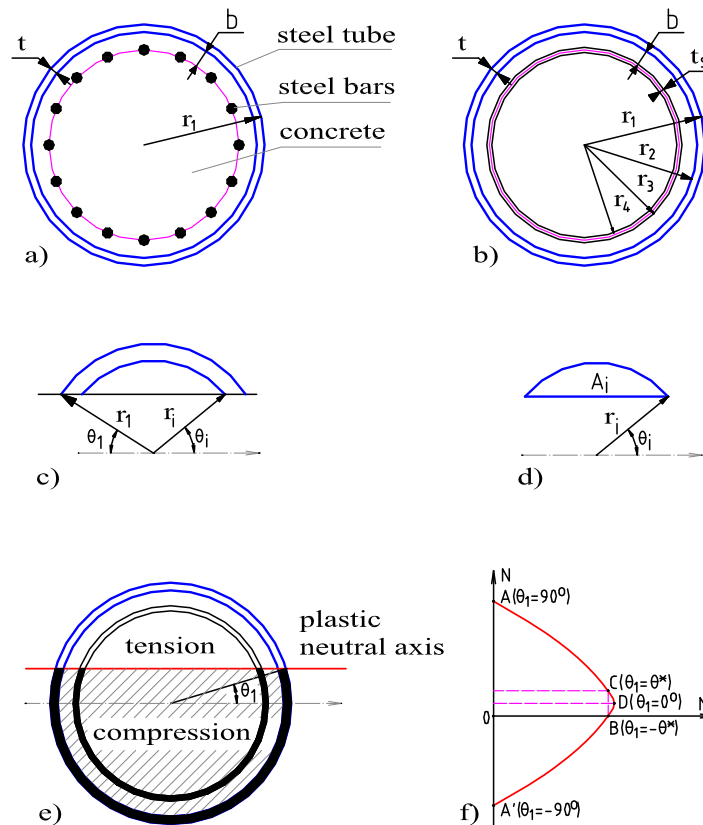


Fig.3: Analysis of composite section

Let  $\theta_1$  vary from  $-\pi/2$  to  $\pi/2$ , using Eqs. (5) and (6), it is possible to depict the interaction curve (from A' to A of Fig.3.f) containing the particular points A, B, C, D (Figs. 3e and 3f):

- Point A corresponds to  $\theta_1 = \pi/2$  ;
- Point B corresponds to  $\theta_1 = \theta^*$  ;  $\theta^*$  is determined by Eq.(5), but  $N_{\theta^*} = 0$  ;
- Point D corresponds to  $\theta_1 = 0$  ;
- Point C is deduced by points B and D.

In the case of no reinforcements,  $A_s = 0$  leading to  $r_3 = r_4$ .

- *Member verification:*

The simplified method allowed by Eurocode-4 is summarized as follows:

Two values for the effective flexural stiffness are distinguished:

$$(EI)_{eff} = E_a I_a + E_s I_s + 0,6 E_{cm} I_c ;$$

$$(EI)_{eff,II} = 0,9(E_a I_a + E_s I_s + 0,5 E_{cm} I_c) ;$$

where  $E_{cm}$  is the concrete modulus of elasticity taking into account the influence of long-term effects, it depends on the values of long and short term loading and the creep coefficient (for reason of simplification,  $E_{cm} = E_c / 1,6$  in the present work);  $I_a, I_s, I_c$  are the respective second moment of inertia of the steel, the rebar and the concrete.

$(EI)_{eff}$  is used to calculate the elastic critical axial compression force and, subsequently, the relative slenderness whereas  $(EI)_{eff,II}$  is used to determine the second-order effect of the member.

(1) The resistance of the member under axial compression is verified using:

$$\frac{N_{Ed}}{\chi N_{pl,Rd}} \leq 1, \quad (7)$$

where  $N_{pl,Rd}$  is the plastic compression resistance, calculated with Eq.(5) in which  $\theta_1 = \pi/2$  ;  $N_{pl,Rk}$  is the characteristic value of  $N_{pl,Rd}$  ;  $N_{cr}$  is calculated with Eq.(2) using  $(EI)_{eff}$  ;  $\bar{\lambda}$  is calculated using  $\bar{\lambda} = \sqrt{N_{pl,Rk} / N_{cr}}$ . The Eq.(3) provides the reduction factor  $\chi$ .

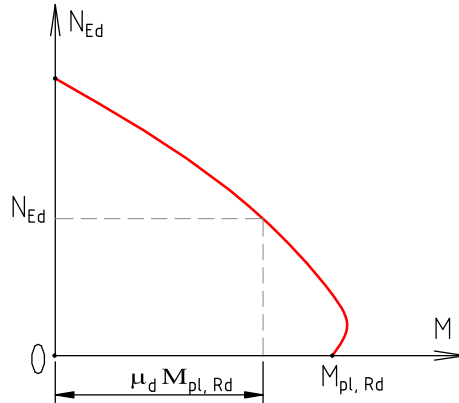
(2) The resistance of the member under compression and bending is verified using:

$$\frac{M_{Ed,II}}{\mu_d M_{pl,Rd}} \leq \alpha_M, \quad (8)$$

where  $\mu_d$  is provided in Fig.4;  $\alpha_M = 0,9$  for NS and 0,8 for HSS and the second-order bending moment  $M_{Ed,II}$  is calculated using:

$$M_{Ed,II} = \left( M_{Ed} + N_{Ed} \frac{L}{300} \right) \frac{1}{1 - N_{Ed} / N_{cr,II}},$$

where  $N_{cr,II}$  is calculated using Eq.(2) together with  $(EI)_{eff,II}$ .



**Fig.4:** Member verification for composite columns

- *Conditions of use of the simplified method:*

To apply the simplified method for circular hollow sections, the following conditions should be satisfied:

- The member is not too slender:  $\bar{\lambda} \leq 2,0$ ;
- The area of rebar shouldn't exceed 6% of the one of concrete core.
- The section belongs to Class 3 at the minimum, in order to avoid the local buckling of the steel tube.

### 2.1.3. Establishment of optimum problem

After having the safety condition of columns, we can build up the optimal problem that may be described as the following.

- *Cost function and unknowns:*

The following parameters may be considered as the variables of the optimal problem:

- The diameter  $D$  and thickness  $t$  for steel columns;
- The diameter  $D$ , thickness  $t$ , distance  $b$  (Fig.1), area of rebar, class of concrete and grade of rebar for composite columns.

However, the class of concrete and the grade of rebar are discontinuous quantities with very few practicable values. And thus they are not considered as regular variables of the problem.

Concerning the distance  $b$ , we can say that, under static loading, the capacity of the section increases if  $b$  decreases, with  $b$  obviously respecting the constitutive condition. We could therefore fix the value of  $b$  at the beginning of the optimum research problem.

Moreover, in order to be able to compare the two solutions, the cost of several quantities (e.g. each grade steel, each grade of rebar and each class of concrete) must be defined taking into account its variability with respect to the time and the country. Since the objective is to draw general

conclusions useful for any time and place, a large field of the mentioned costs should be investigated, obviously leading to the complexity of the problem. To avoid this, the following problem for composite columns is considered: two solutions of columns are compared with the same length, the same class of concrete, the same density (%) of rebar, under the same load, but using two different values of strengths of steel tubes. The variations of length, loads, concrete class, and rebar density will be considered as the parameters (*input variable*) of the optimum research problem. Therefore, the following cost function is adopted:

$$C = l(A_a c_a + A_{cs} c_{cs}), \tag{9}$$

where  $A_{cs}$  is the area of concrete and rebar;  $c_a$ ,  $c_{cs}$  are, respectively, the cost per volume of steel and of reinforcement concrete (euros/m<sup>3</sup>).

Meaning that, when calculating the cost, the concrete and rebars are considered as one single material (*reinforced concrete*). The parameter  $c_{cs}$  obviously depends on the class of concrete and the density of rebar.

Finally, two variables have to be examined: the diameter  $D$  and the thickness  $t$ . In reality, market catalogues for steel tubes provide discontinuous quantities for the couple  $D$  and  $t$ . But, in the present research, in order to generalize the results and simplify the mathematical problem, the diameter  $D$  and the thickness  $t$  are considered as continuous quantities.

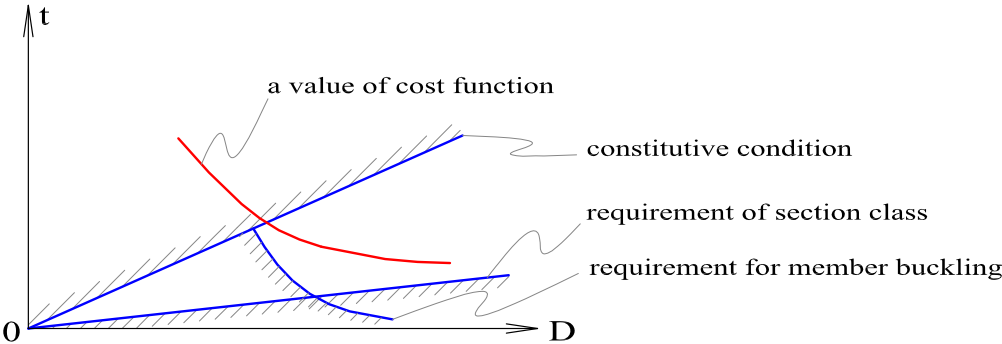
- Constraints:

Using the safety analyses of columns presented in Sections 1 and 2, the constraints for the optimum research problem are summarized as follows:

- Requirement of *section classification*: Eq.(1);
- Requirement for *member buckling resistance*: Eq.(4) for steel columns, Eqs.(7) or (8) for composite columns;
- *Constitutive condition*:  $t \leq D / 2$ .

- Geometric interpretation of the optimum research problem:

The optimum research problem can be qualitatively interpreted as it is depicted in Fig.5.



**Fig.5:** Geometric interpretation of the optimum research problem for simple columns

- Mathematical procedure

Mathematically, the problem can be written under the form:

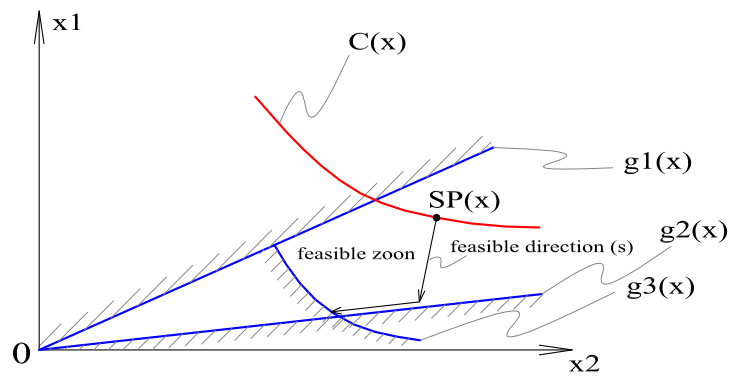
$$\text{Find } \mathbf{x} = [D, t] \text{ such that } C(\mathbf{x}) \rightarrow \min \text{ but } g_j(\mathbf{x}) \leq 0, j = 1 \div n, \quad (10)$$

where  $C(\mathbf{x})$  is the cost function (Eq.(9));  $g_j(\mathbf{x})$  are the constraints.

The method of *feasible direction* is chosen to solve the problem. The explanation of this method is abundantly reviewed in the literature (e.g. [7]). Herein, the main ideas are briefly recalled: (1) an initial point SP is found inside the *feasible zoon* (Fig.6); (2) a *feasible direction*  $\mathbf{S}$  is established, with which a new point considered as better than the last point (the cost function decreases with the constraints still respected) is found; (3) the allowed distance in the direction  $\mathbf{S}$  is limited by a scalar  $\alpha^*$ . The procedure is repeated until an acceptable convergence (optimum) is reached, it is the case when no feasible direction is found.

The  $q^{\text{th}}$  iteration of the process can be written as:

$$\mathbf{X}^q = \mathbf{X}^{q-1} + \alpha^* \mathbf{S}^q$$



**Fig.6:** Illustration of the optimum research problem procedure

In the procedure, it is necessary to calculate the gradient of  $\mathbf{C}$  and  $\mathbf{g}$  although it is sometimes difficult to compute the derivative of these functions such that the gradient is often replaced by the sensitivity:

$$\frac{\partial \mathbf{g}(x_i)}{\partial x_i} \approx \frac{\mathbf{g}(x_i + \Delta x_i) - \mathbf{g}(x_i)}{\Delta x_i}$$

## 2.2. Definition of the index of interest

At the present time, it seems that the grade of steel S355 is the most popular in construction, it is thus chosen as the *reference material*. Eq.(9) (with the sub-scripts “355” and “HSS” to distinguish the reference steel and HSS,

$$C_{355} = l(A_{a,355}c_{a,355} + A_{cs}c_{cs});$$

$$C_{HSS} = l(A_{a,HSS}c_{a,HSS} + A_{cs}c_{cs});$$

can be rewritten as

$$C_{HSS} / C_{355} = \frac{(A_{a,HSS} c_{a,HSS} / c_{355} + A_{cs,HSS} c_{cs} / c_{355})}{(A_{a,355} + A_{cs,355} c_{cs} / c_{355})}. \quad (11)$$

It is clear that if  $C_{HSS} / C_{NS} < 1$ , then HSS is of interest; on the contrary, if  $C_{HSS} / C_{NS} > 1$  then NS is of interest; the neutral case occurs if  $C_{HSS} / C_{NS} = 1$ .

### 2.3. Field of investigation for simple columns

In order to draw conclusions that might be true for a lot of practical cases, the following fields are investigated.

- the columns length / varies from 3 to 8 m;
- the compression force  $N_{Ed}$  varies from 500 to 6000 kN;
- the maximum bending moment  $M_{Ed,max}$  to compression force  $N_{Ed}$  ratio varies from 0 to 0.75 m;

- according to [4], S500, S550, S620 and S690 steels have been considered as HSS (with  $f_y = 500, 550, 620$  and  $690$  (N/mm<sup>2</sup>) respectively). In the present work, various steel grades within 500 and 700 are considered. However, in Appendix, in order to limit the number of charts, three supposed HSS steels are examined: S500, S600 and S700; the results concerning an intermediate steel grade could be interpolated using the results of the other steel grades.

- the characteristic value of the compressive concrete cylinder strength varies between  $f_{ck} = 25 \div 40$  (N/mm<sup>2</sup>); the density of rebar varies from 0% to 6%;

- the cost of HSS to cost of S355 ratio  $c_{HSS} / c_{355} = 1,1 \div 1,6$ . According to [1], these values (interpolated using Fig.1 in [1]) are:  $c_{500} / c_{355} = 1,138$ ;  $c_{550} / c_{355} = 1,260$ ;  $c_{620} / c_{355} = 1,340$ ;  $c_{690} / c_{355} = 1,382$ .

- the cost of reinforced concrete to the cost of S355 ratio  $c_{cs} / c_{355} = 0,02 \div 0,05$ . At the moment, this value in Belgium is around 0,03.

### 2.4. Numerical results and comments for simple columns

In order to illustrate the procedure presented in the above sections, an example is provided here below for a 5m length column, submitted to a compression load of 5000 kN and a uniform bending moment of 100 kNm. The costs of two columns are compared:

- (1) Steel column made of S355 and S690,
- (2) Composite column using an outer steel tube made of S355 and S690, with inside concrete C30/37 ( $f_{ck} = 30$  N/mm<sup>2</sup>,  $f_{cd} = 20$  N/mm<sup>2</sup>) reinforced with a density of rebar equal to 4% ( $f_{sy} = 500$  N/mm<sup>2</sup>).

The summarized results (geometry and costs) are:

- (1) For the steel column:

- Optimal solution for column using S355: D=49,97 cm; t=1,08 cm.

- Optimal solution for column using S690: D=33,73 cm; t=1,10 cm.

- Comparison of costs:

The ratio  $c_{355}A_{355}/c_{690}A_{690}=1,46 c_{355}/c_{690}$ , meaning that the use of HSS can provide an economic interest. Let's consider  $c_{690}/c_{355}=1,382$  [7], in this case, the economic interest of using S690 compared to S355 would be 5,6%.

(2) For the composite column:

- Optimal solution for column using S355: steel tube D=40,02 cm, t = 0,67 cm;  $A_s = 45,17\text{cm}^2$  (= 4% of the concrete area).

- Optimal solution for column using S690: steel tube D=31,61 cm, t = 1,03 cm;  $A_s = 26,38\text{cm}^2$  (= 4% of the concrete area).

- Comparison of costs:

Using Eq.(11), one has:

$$\frac{\text{cost of column using S690}}{\text{cost of column using S355}} = \frac{98,95c_{690}/c_{355} + 685,8c_{cs}/c_{355}}{81,42 + 1175,1c_{cs}/c_{355}}$$

If  $c_{690}/c_{355}=1,382$  is adopted again, the use of S690 is of interest if the *unrealistic* condition  $c_{cs}/c_{355} \geq 1/8,8$  is respected. In this case, HSS does not provide any economic interest.

After all calculations achieved in the chosen filed of applications, it is possible to draw several conclusions:

- In many case, for steel columns, the use of HSS leads to considerable economic profit in comparison with S355 steel. In fact, the use of HSS in case of stocky columns provides the greatest advantage while NS is more economic in case of slender columns. Moreover, the interest of using HSS decreases when the eccentricity increases.

- Depending on the column length and the loading condition (M/N), the charts A1 to A12 in the Appendix show the ratio between the area of HSS columns and the area of NS columns (for simple columns). With these charts, the user can obtain the economic benefit of the use of HSS if the material costs are known. For reason of simplification, only a few charts are presented.

- Even if the relative cost  $c_{cs}/c_{355}$  is varying a lot, very few of cases where the use of HSS in composite columns gives economic profit. By way of conclusion, it is not economic to use HSS tubes for composite columns under static loading.

### 3. Investigation for columns in frames

The simple column, studied in Section 2, constitutes a relatively unrealistic case. Indeed, in reality, the columns are connected with other members composing the structure. The pin-end boundary conditions occurring in the case of single columns are rather ideal. While, in the case of columns included in frames, an interaction exists with the rest of the structure, meaning that the main difference between the two cases are the boundary conditions. Figures 7a and 7b are traditionally used to represent the column in *braced frames* and *un-braced frames* respectively.

Similarly to simple columns, the above described procedure is carried out to compare two solutions of column using NS and HSS.

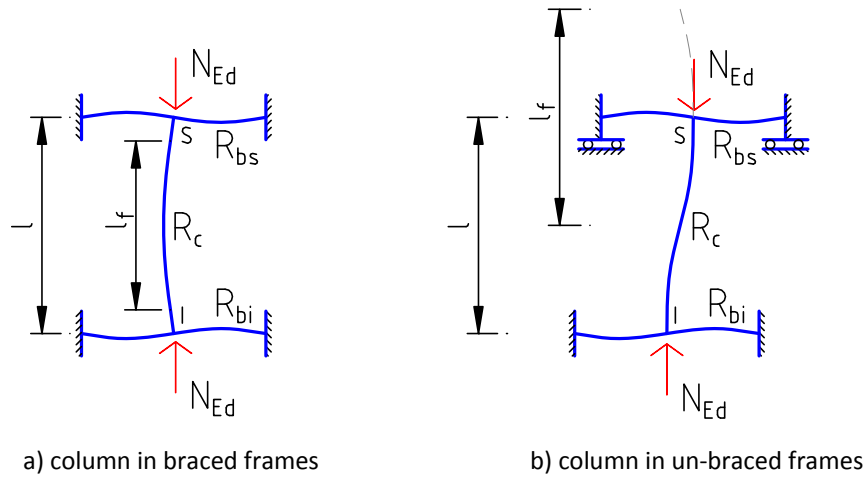


Fig.7. Column in frames

### 3.1. Analysis of columns in frames

#### 3.1.1. Effective length

Traditionally, the concept of effective length has been used to evaluate the stability of columns in frames, using the analogy of the simple column of same length. Wood's research [11] on the effective length is adopted in the present work. According to this, the effective length depends on the stiffness coefficients at column crossings:

$$k_s = \frac{R_c + R_s}{R_c + R_s + R_{bs}}; \quad (12)$$

$$k_i = \frac{R_c + R_i}{R_c + R_i + R_{bi}}; \quad (13)$$

where  $R_c$  is the stiffness of the considered column;  $R_s$  and  $R_i$  are the stiffness of the upper and lower columns respectively;  $R_{bs}$  and  $R_{bi}$  are respectively the sum of the stiffness of all beams connected at the superior node S and the inferior node I of the considered column (Fig.7).

With  $k_s$  and  $k_i$ , we can obtain the effective length of the column using the charts that were developed by Wood [11]. However, applying the charts is not very suitable to the automatic computation that is required in the present research, such that the following formulas Eqs.(14) and (15), approximating Wood's charts, are preferred (see [2]).

$$l_f = l \left[ \frac{1 + 0,145(k_s + k_i) - 0,265k_i k_s}{2 - 0,364(k_s + k_i) - 0,247k_i k_s} \right] \text{ for braced columns} \quad (14)$$

$$l_f = l \left[ \frac{1 - 0,37(k_s + k_i) + 0,01k_i k_s}{1 - 0,9(k_s + k_i) + 0,8k_i k_s} \right]^{1/2} \text{ for un-braced columns} \quad (15)$$

As soon as the effective length is known, the stability analysis of columns in frames is similar to the one of single columns.

### 3.1.2. Horizontal displacement of column in un-braced frames

The stiffness of columns made of HSS is smaller than the one of columns made of NS, such that the horizontal displacement might become important. Taking into account the displacement in the optimal problem for columns in un-braced frames is necessary in order to have realistic results. The horizontal displacement of the column in un-braced frames due to the horizontal load (Fig.7) is calculated using the following formula [8]

$$\Delta = \frac{P_k l^3}{12EI_c} \left( 1 + \frac{3(k_i + k_s - k_i k_s)}{4 - 3k_i - 3k_s + 2k_i k_s} \right),$$

where  $E$  is the modulus of plasticity for steel/composite column;  $P_k$  is the characteristic value of the horizontal load (Fig.8).

If the second-order effects are taken into account, the displacement becomes

$$\Delta = \frac{P_k l^3}{12EI_c} \left( 1 + \frac{3(k_i + k_s - k_i k_s)}{4 - 3k_i - 3k_s + 2k_i k_s} \right) \left( \frac{1}{1 - N_{Ek} / N_{cr}} \right),$$

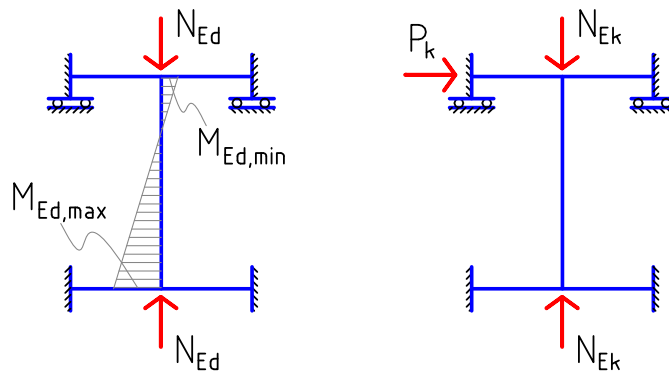
with  $N_k$  is the characteristic value of the vertical load (Fig.8).

### 3.2. Optimum problem for columns in frames

If columns in braced frames, the optimum problem is similar to the one if simple columns. A procedure for calculating the effective length using Eqs.(14) and (15) is simply added. The load combination shown on Fig.8a is used for the stability study, achieved using Eqs.(4), (7) and (8).

If columns in un-braced frames, besides this new procedure, another limitation has to be taken into account: the horizontal displacement of the columns must remain under  $l/250$ , where  $l$  is the column length. For that case, the load combination reported on Fig.8b is adopted and must respect the displacement condition written as

$$\Delta = \frac{P_k l^3}{12EI_c} \left( 1 + \frac{3(k_i + k_s - k_i k_s)}{4 - 3k_i - 3k_s + 2k_i k_s} \right) \left( \frac{1}{1 - N_{Ek} / N_{cr}} \right) \leq l / 250 .$$



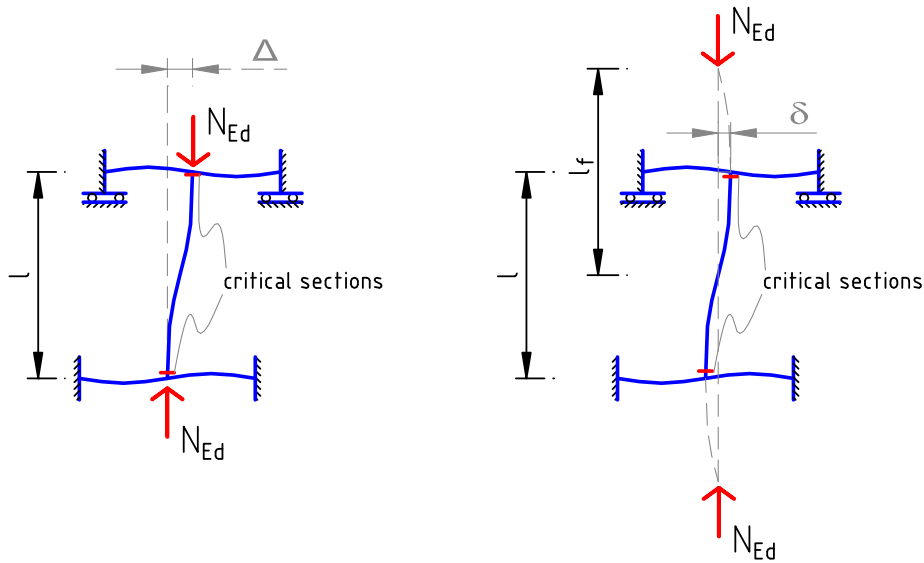
a) Ultimate state

b) Serviceability limit state

**Fig.8.** Two states to be verified for column in un-braced frames

For the columns in un-braced frames, the equivalent uniform moment factor  $C_m = 1$  should be adopted since the extremities of the column are the critical parts to be considered in the stability problem (Fig.9). In this case, it is not necessary to verify the cross-sections strength condition.

Moreover, it is worth noting that, if the effective length is calculated using the mentioned equations (14) and (15), the P-  $\Delta$  effect is taken into account. And therefore, the bending moment  $M_{Ed,max}$  (Fig.8a) should be calculated using the first order theory.



**Fig.9.** Critical sections for columns in un-braced frames

### 3.3. Field of investigation for columns in frames

For the two types of column shown in Fig.7, the field of investigation of the simple column (Section 2.3) is reused. Additionally, the variation of the coefficients  $k_i$  and  $k_s$  and of the horizontal load have to be taken into account. Besides, the *characteristic* value of the axial load has to be considered (Fig.8b). In order to be able to compare two solutions of columns (using HSS and NS) and decrease the complexity of the problem, the following assumptions are made:

- The stiffness at the bottom and top ends of the column are the same  $R_s = R_i = R_c$  ;
- The same configuration of beams is used in every case:  $R_{bs} = R_{bi} = R_b$  .

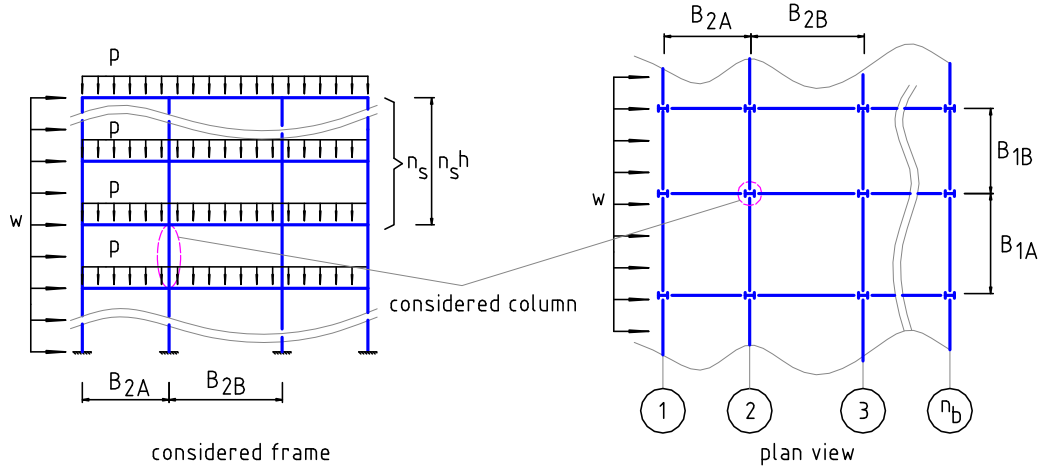
The equations (12) and (13) become:

$$k_s = k_i = k = \frac{2R_c}{2R_c + R_b}, \quad (16)$$

in which k varies from 0 to 1.

- The characteristic value of the vertical load is approximated as  $N_{Ek} = N_{Ed} / 1,4$  ;

- The ratio  $P_k / N_{Ek}$  varies from 1/250 to 1/2. This can be explained by the fact that if one considers a frame of  $n_s$  stories and  $n_b$  bays as the one shown in Fig.10. Prior to any calculation, the approximately estimations of the vertical load and the horizontal load acting on the considered column are:



**Fig.10.** Evaluation of the loads acting on columns

$$N = n_s p \frac{B_{1A} + B_{1B}}{2} \frac{B_{2A} + B_{2B}}{2} ;$$

$$P = w \frac{B_{1A} + B_{1B}}{2} n_s h \frac{1}{n_b} .$$

Consequently, one has:

$$P / N = \frac{w}{p} \frac{h}{n_b \left( \frac{B_{2A} + B_{2B}}{2} \right)} . \quad (17)$$

In traditional buildings, the following limitations are used (units are kN and m):  $10 \leq w \leq 20$ ;  $50 \leq p \leq 120$ ;  $2 \leq n_b \leq 8$ ;  $3 \leq h \leq 5$ ;  $6 \leq B_{2A} + B_{2B} \leq 16$ . These limitations introduced in Eq.(17) provide us with the suggested upper and lower bounds.

### 3.4. Comparison procedure for columns in frames

The same system of beams is supposed to be used for the two cases of column, meaning that we consider a given set of beams and want to compare the optimal NS and HSS columns in this building.

Nevertheless, the stiffness of the columns is not the same leading to different coefficient  $k$ . If  $k$ 's value is chosen for one column type, it is an unknown for the other column type. Therefore, the following procedure is adopted to solve the problem:

*Step 1:* Assign a value to  $k$  (the values varies from 0 to 1 with a step of 0,1) and calculate the effective length according to Eqs.(14) and (15), the column is now considered as a simple column.

*Step 2:* Achieve the optimum research problem (Section 3.2) for the column made of steel S355.

*Step 3:* Calculate  $R_c$  for this optimal column section.

*Step 4:* Determine the value of  $R_b$  using Eq.(16) and the previously calculated  $k$  and  $R_c$ .

*Step 5:* This value of  $R_b$  is considered as an input of the optimum research problem for HSS column. It is worth pointing that  $R_c$  and  $k$  are, of course, updated during the procedure such that Eq.(16) is satisfied.

*Step 6:* The comparison of the two types of column can be made (Section 2.2).

### 3.5. Numerical results and comments for columns in frames

In order to clarify the comparison procedure, an example is presented in details: Let's consider a column in an un-braced frame with a length of  $l = 4\text{m}$ , submitted to  $N_{Ed} = 2000\text{ kN}$  and  $P_k = 33\text{ kN}$  (see Fig.8). One compares the price of two columns made of S355 and S690 grades.

*Step 1:* Let consider  $k=0.2$  for instance, the effective length is calculated using Eq.(15):  $l_f = 450,5\text{ cm}$ .

*Step 2:* The optimum research problem is solved for a column made of S355:  $D_{op,355} = 36,14\text{ cm}$ ;  $t_{op,355} = 0,78\text{ cm}$ ;  $A_{355} = 86,65\text{ cm}^2$ ; the elastic buckling load  $N_{cr,355} = 13480\text{ kN}$ ; the stiffness  $R_{c,355} = 8108\text{ kNm}$ ;  $N_{Ed} / N_{cr,355} = 6,92$  (sway column) and the relative horizontal displacement  $\Delta / l = 1 / 418$ .

*Step 3:* With the optimal section:  $R_c = 7113,15\text{ kNm}$ .

*Step 4:* The stiffness of the contiguous beams is calculated using Eq.(16):  $R_b = 56918\text{ kNm}$ .

*Step 5:* Then, the optimum research problem is solved for a column made of S690 and characterized by the same configuration of beams ( $R_b = 64862\text{ kNm}$ ):  $D_{op,690} = 28,60\text{ cm}$ ;  $t_{op,690} = 0,93\text{ cm}$ ;  $A_{690} = 81,10\text{ cm}^2$ ; the elastic buckling load  $N_{cr,690} = 8725\text{ kN}$ ; the stiffness  $R_{c,690} = 4077\text{ kNm}$ ; the coefficient  $k_{690} = 0,125$ ; the effective length  $l_{f,690} = 430\text{ cm}$ ;  $N_{Ed} / N_{cr,690} = 4,36$  (sway column) and the relative displacement  $\Delta / l = 1 / 250$ .

*Step 6:* Comparison: one has  $c_{355}A_{355} / c_{690}A_{690} = 1,07c_{355} / c_{690}$  thus the use of S690 is of interest if  $c_{690} / c_{355} \leq 1,16$ . Although this conclusion depends on the market price, we can say that there are very few possibilities where S690 is interesting in this case.

A large number of cases covering the field of investigation (Section 3.3) are computed. For reason of clarity, only a limited amount of charts are chosen to show the volume reduction of HSS columns in comparison with NS columns are reported in the Appendix. Generally, the following comments and conclusions can be drawn:

- If braced frames and steel columns, the domain of interest of HSS (positive economic profit) is greater than the one for simple columns. This can be explained by the fact that the simple column effective lengths (being the column length) are the same for NS and HSS columns. On the contrary, the effective length of the column in braced frames (with the same beam system) is smaller if HSS is used instead of NS. Even if the bending moments in braced frame are not significant in practice but different values of the eccentricity are considered in the present work.

- The economic benefit of the use of HSS in un-braced frames is smaller than the one in the case of braced frames. The displacement condition is responsible of this, emphasizing the disadvantage of the use of HSS. The volume reductions are also shown in the Appendix.

- Generally, the terms *sway* (if the vertical to elastic critical load ratio  $N_{Ed} / N_{cr} < 10$ ) and *non-sway* frames are used for frames. It is not convenient to use this kind of classification for a comparison of the economic interest, but because these terms are widely used, the following comment has been drawn looking at the numerical results: there is no benefit in using HSS in sway frames with  $N_{Ed} / N_{cr} < 10$  for a comparison made on the basis of frames using S355 steel.

- As it is the case for single columns, there are very few possibilities of composite columns for which the use HSS tubes provide an economic profit.

#### 4. Investigation for frames

To design a frame, the following quantities are necessary: vertical loads, horizontal loads, frame configuration (number of stories, number of span, height of stories...) and technology conditions, etc. It seems unfeasible to consider a certain amount of frames by varying the mentioned quantities and cover almost all practical possibilities. Therefore, even if the optimum design problem could be defined (taking into account the strength, stability and stiffness conditions) and solved using appropriate computational software, the resulting charts wouldn't be similar to the ones obtained in the case of isolated columns.

Nonetheless, the comments of Section 3.4 are also useful for frames. It is the reason why two simplified procedures are proposed here below to help decision-making regarding the steel grade, before any detailed design of the frame be carried out.

*The first one:* This method consists of the following steps: Firstly, prior to any computation, the engineer's expertise leads him to first choose the member sizes of the frame. Afterwards, a global analysis of the frame is introduced in two cases: (1) the first case aim to find the critical internal forces in the columns (design values); (2) in second case we apply only horizontal load with characteristic value to obtain this kind of load distributing in each column. Finally, with the mentioned internal forces and horizontal loads, using the charts provided in the Appendix we could assess the economic interest of the use of HSS.

*The second one:* In this method, we consider each floor separately and choose a column to be considered as the representative column. It is worth choosing an *average* column instead of the overloaded one. The procedure is summarized in three steps: (1) firstly, the vertical load and horizontal load are calculated for the considered floor; (2) next, the loads acting on the representative column are determined, for that purpose the total loads acting on the floor is divided by the number of columns; (3) the charts provided in the Appendix could be used to assess the

economic interest of the use of HSS. This method is more straightforward but less accurate than the first one.

## 5. Conclusion

The economic interest of the use of HSS circular tubes in steel and composite columns submitted to static loading is investigated in the present research paper. The general idea is to compare the costs of columns made of HSS and NS. In order to find comparable designs in each category, the optimum research problem is defined such that the cost of the column is minimum. The realist aspect of the results is confirmed using the current Eurocodes [3, 4, 5] rules for checking the safety of the structures. From simple columns to columns in braced and un-braced frames are considered. A large field of investigation covering almost all possibilities is examined by an automatic algorithm leading to rather general conclusions.

For each investigated case, several charts are provided depending on the loading condition  $M/N$ . In one chart, depicting the length to compression load curves, the user is able to determine the *required area of HSS to required area of NS* ratio for his column. And depending on the material cost of the market, the user can define the *domain of interest* of the use of HSS. For reason of simplification, only a few charts are presented in Appendix. And for whole frames, tracks are provided to help decision-making process.

It is worth pointing out several conclusions for steel simple columns: (1) in many case, the use of HSS leads to considerable economic profit in comparison with S355 steel, especially in case of stocky columns for which the greatest advantage is observed; (2) the interest of using HSS decreases when the eccentricity increases.

And for steel columns in frames: (1) the domain of interest of the use of HSS in braced frames is thought to be relatively large; (2) the economic benefit of the use of HSS in un-braced frames is smaller than the one in the case of braced frames, the displacement condition is responsible of this, emphasizing the disadvantage of the use of HSS; (3) generally, there is no benefit in using HSS in sway frames compared to frames using S355 steel.

In any case (isolated column or columns in frames), even if the relative cost  $c_{cs} / c_{355}$  is varying a lot, there are very few possibilities of composite columns for which the use HSS tubes provide an economic profit.

Finally, the methodology is thought to be applicable for other problems, such other section shapes for instance (e.g. I/H shape).

# Appendix

In this appendix, the charts show the volume reduction of HSS (S500, S600, S700) in comparison with S355, in other words:  $A_{HSS} / A_{NS}$ . The horizontal axis represents the column lengths (m) and the vertical axis provides the compression load (t). The title of each chart informs the user about:  $M_{Ed} / N_{Ed}$  (replaced by M/N) and  $P_k / N_{Ek}$  (replaced by P/N, for reason of clarity). The charts are arranged in the following order:

- Figs. A1 - A12 for simple columns;
- Figs. A13 - A24 for columns in braced frames, with  $k=0,3$ ;
- Figs. A25 - A72 for columns in un-braced frames, with  $k=0$ .

The domain where the economic profit could be expected is always represented by the group of lines in cold colours.

The user can also interpolate the results for its own convenience or may request the authors for more information.

## Charts 1 to 12. Single columns

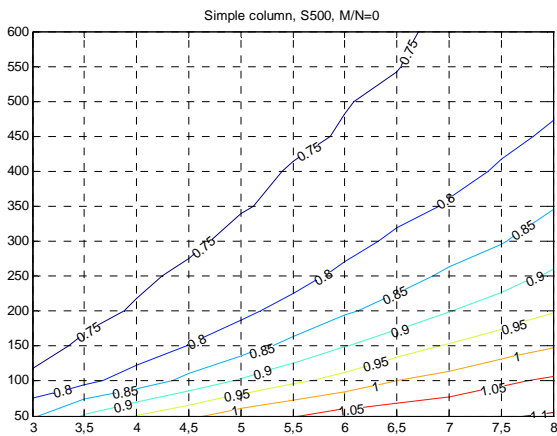


Fig.A1

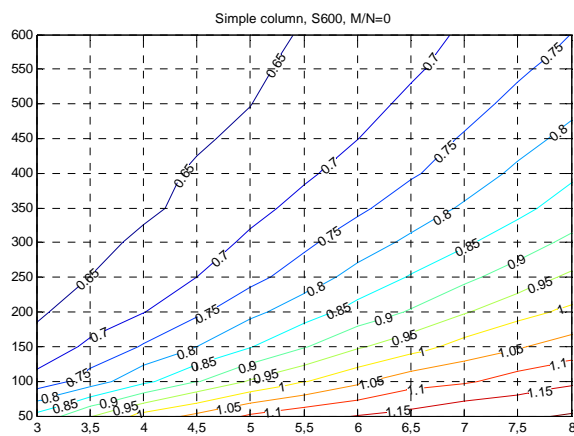


Fig.A2

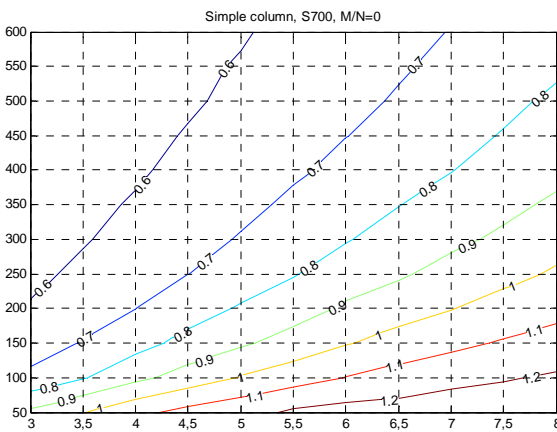


Fig.A3

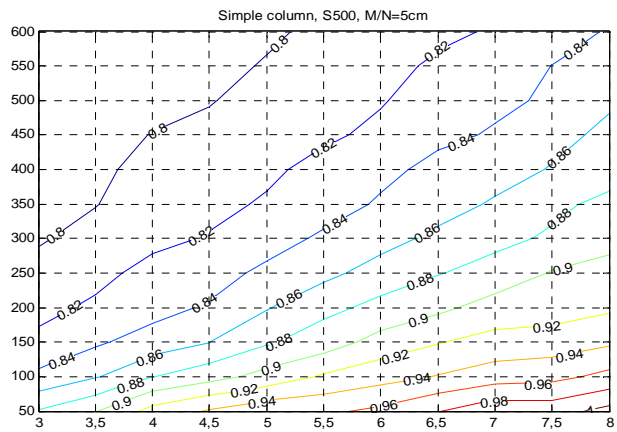


Fig.A4

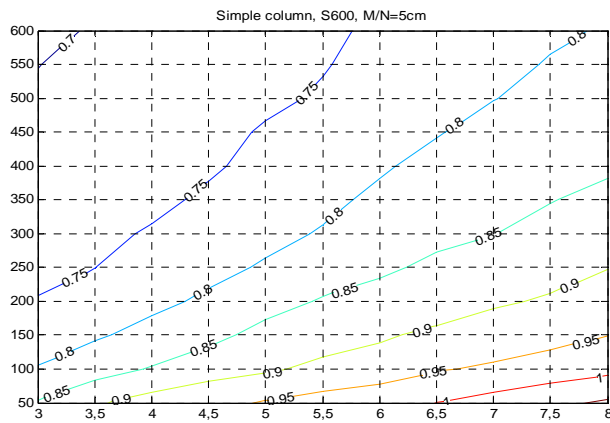


Fig.A5

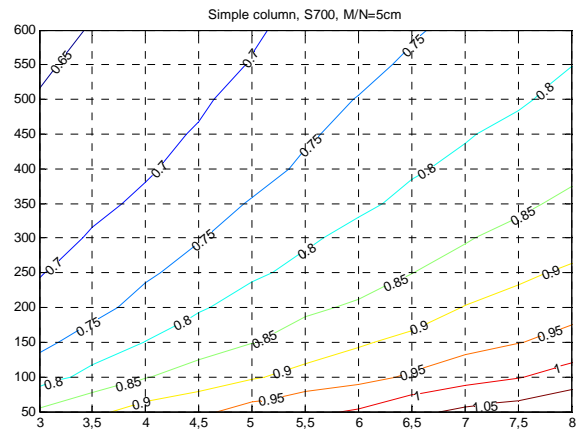


Fig.A6

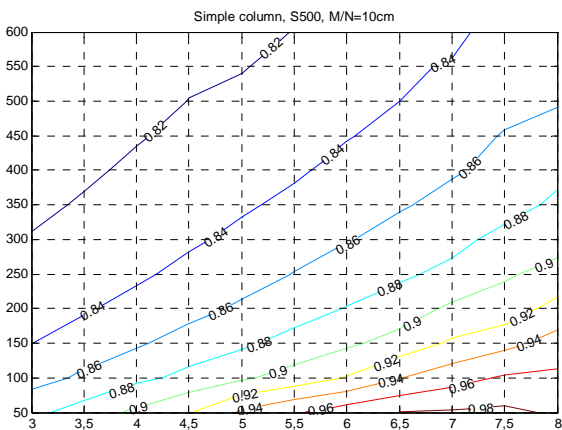


Fig.A7

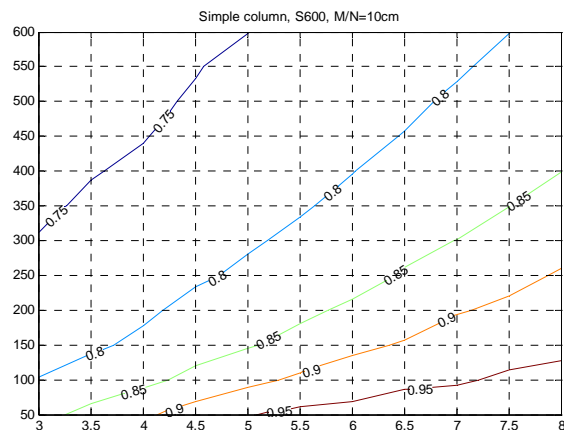


Fig.A8

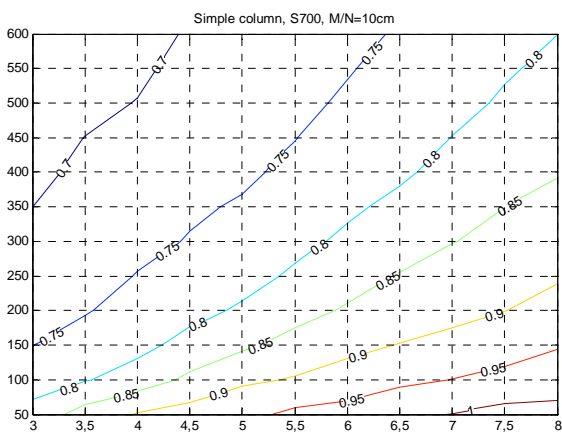


Fig.A9

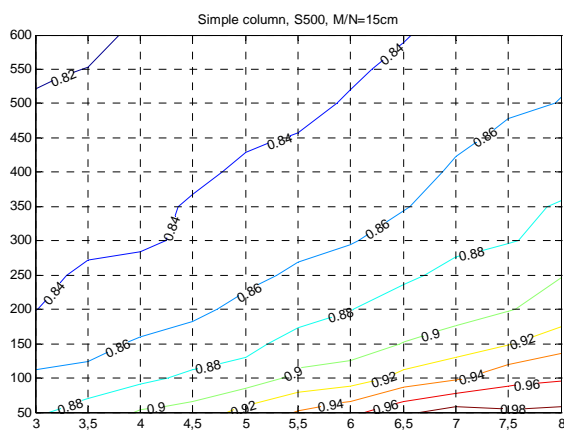


Fig.A10

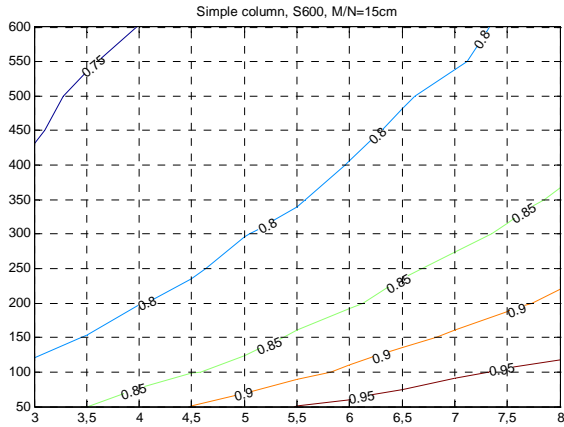


Fig.A11

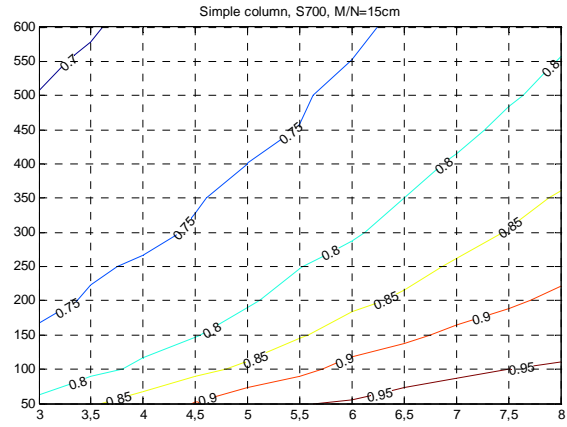


Fig.A12

Charts 13 to 24. Columns in braced frames

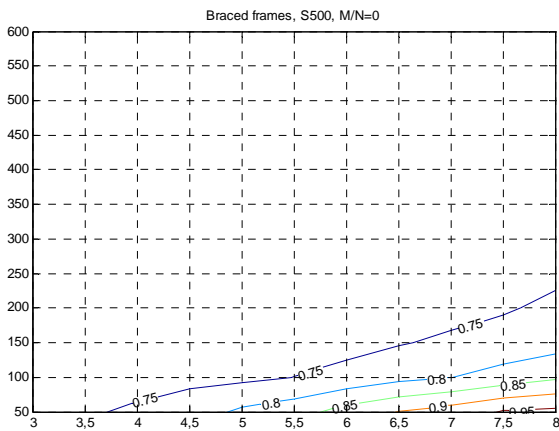


Fig.A13

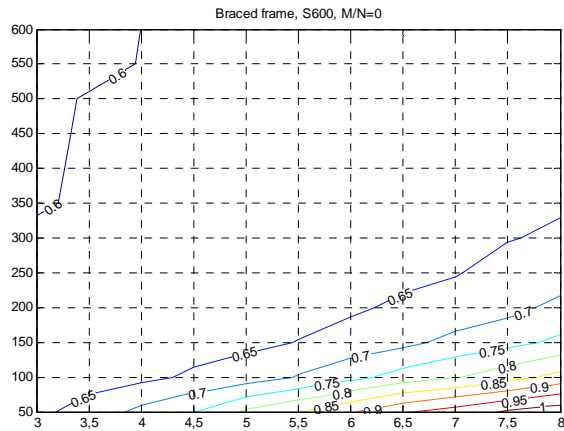


Fig.A14

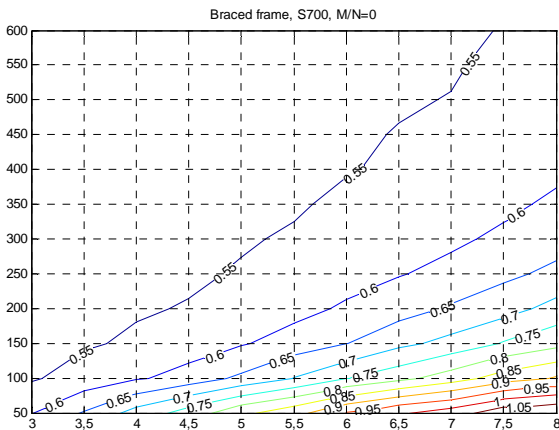


Fig.A15

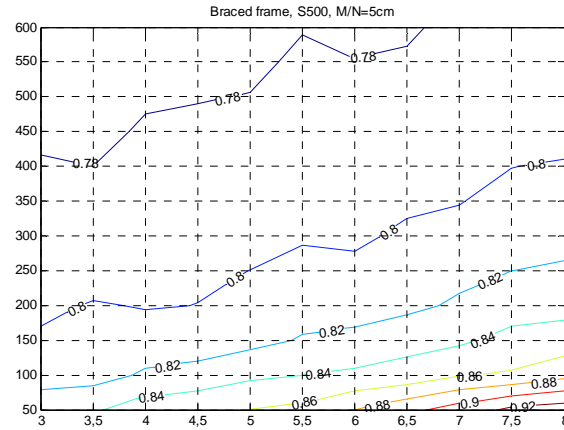


Fig.A16

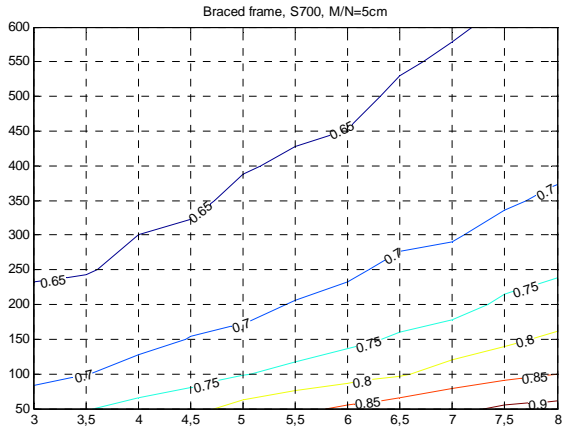


Fig.A17

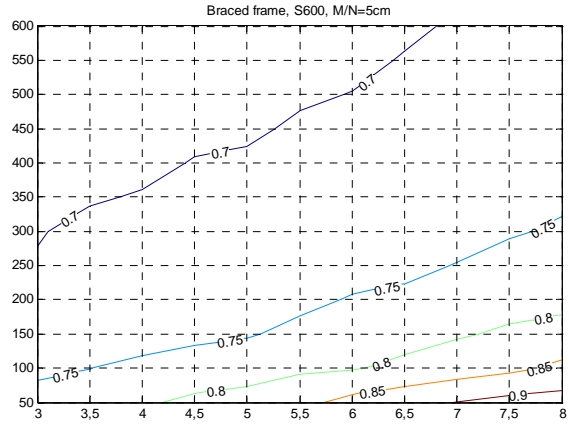


Fig.A18

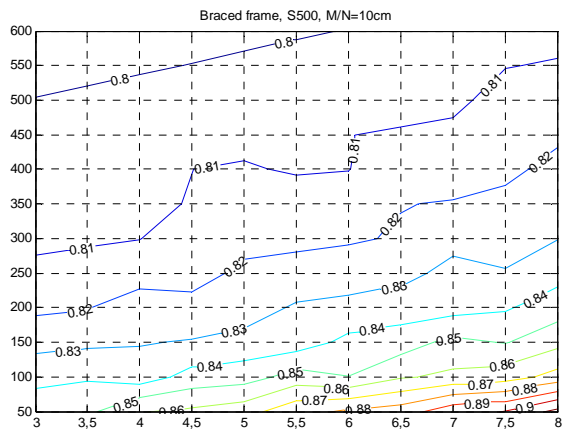


Fig.A19

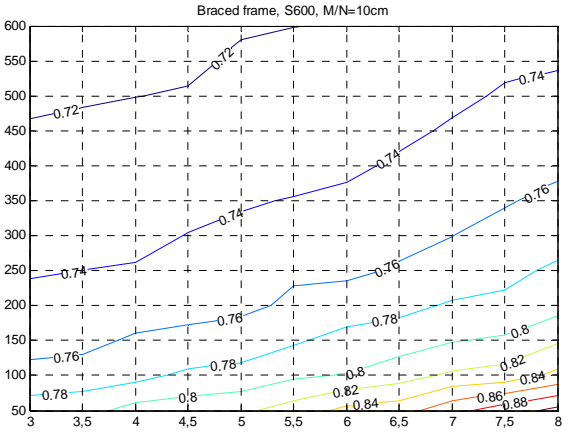


Fig.A20

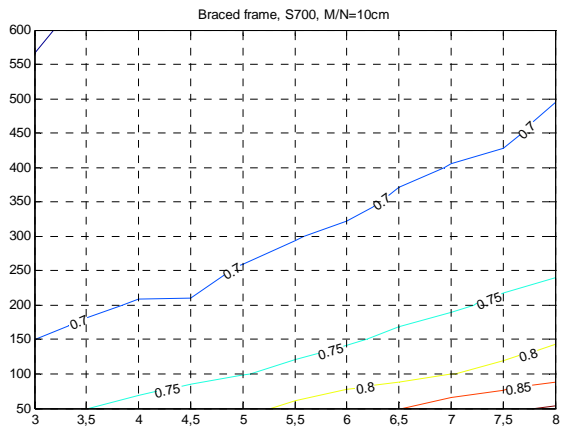


Fig.A21

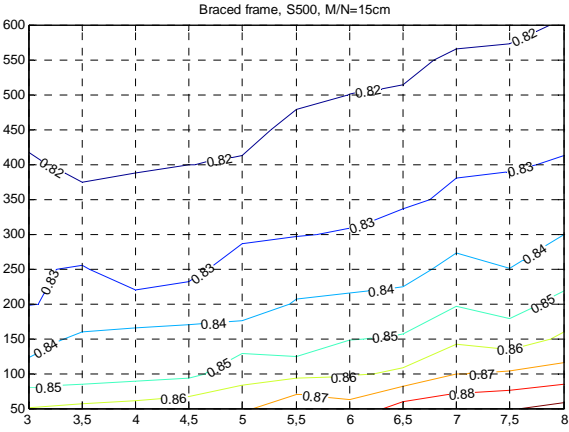


Fig.A22

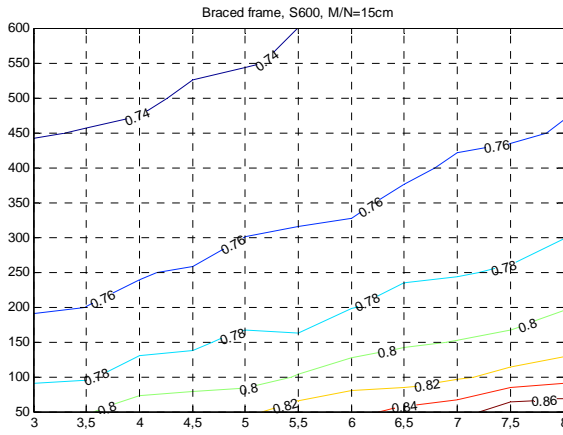


Fig.A23

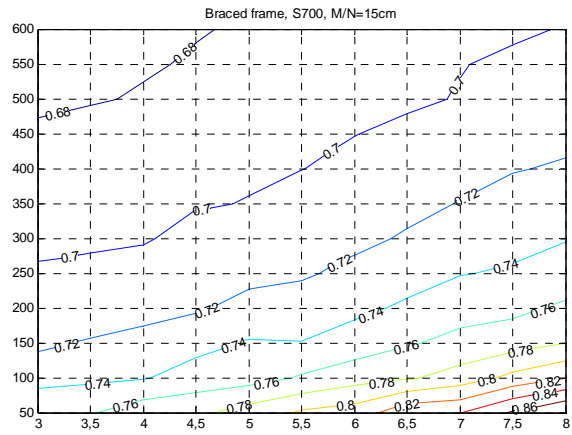


Fig.A24

Charts 25 to 72. Columns in braced frames

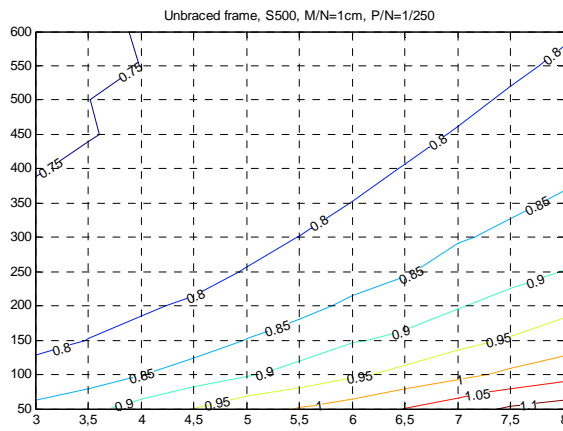


Fig.A25

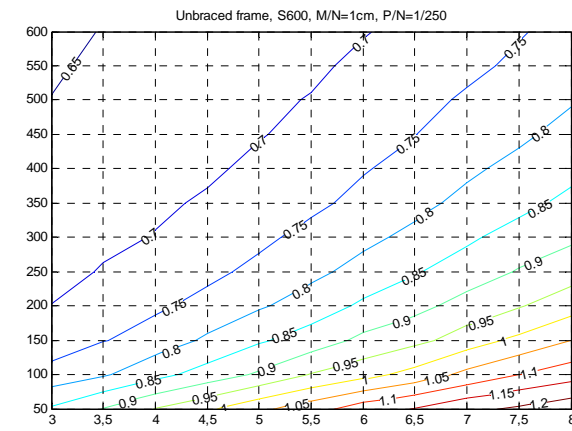


Fig.A26

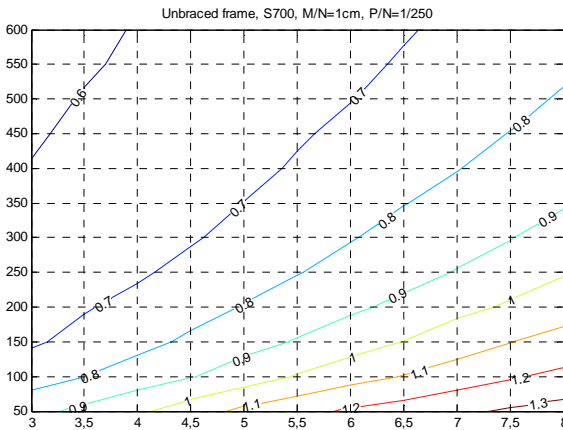


Fig.A27

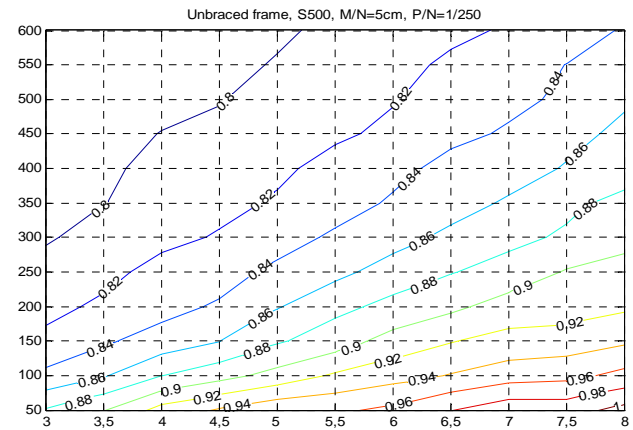


Fig.A28

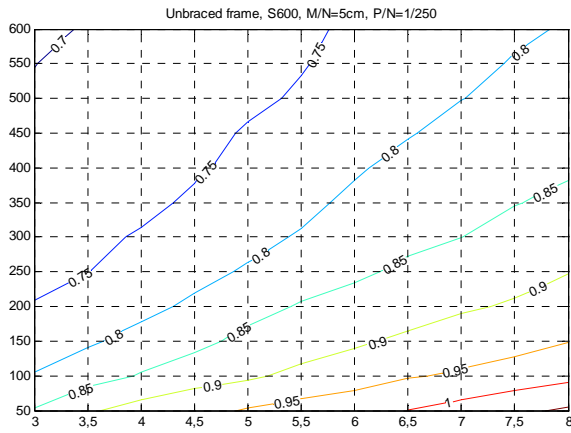


Fig.A29

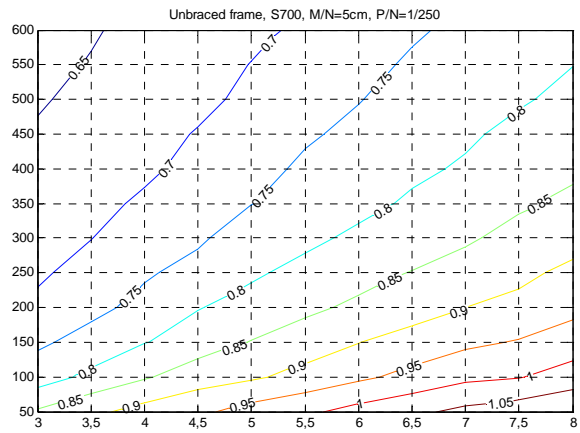


Fig.A30

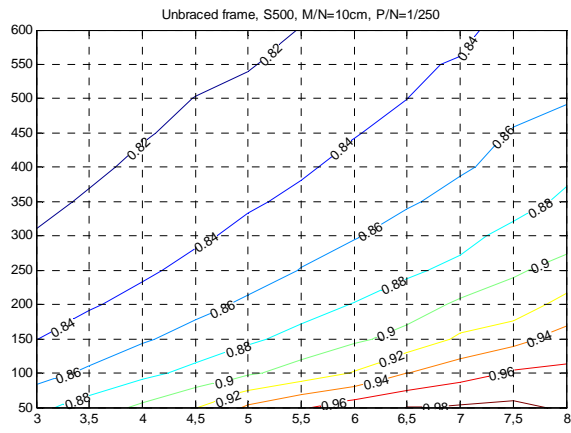


Fig.A31

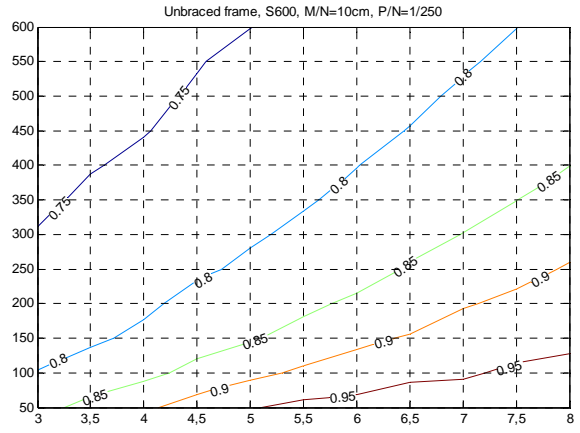


Fig.A32

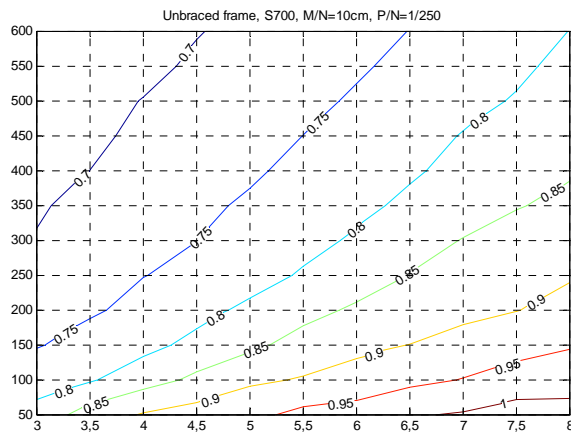


Fig.A33

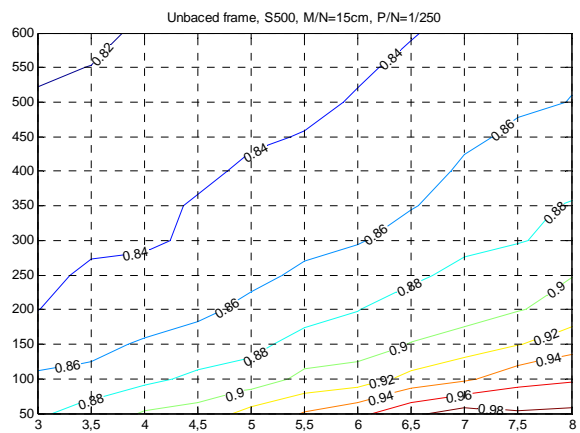


Fig.A34

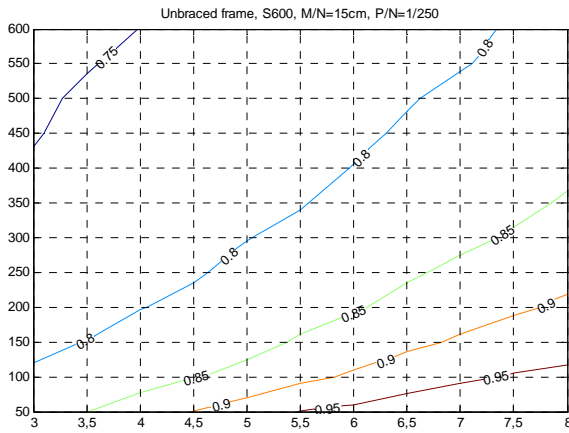


Fig.A35

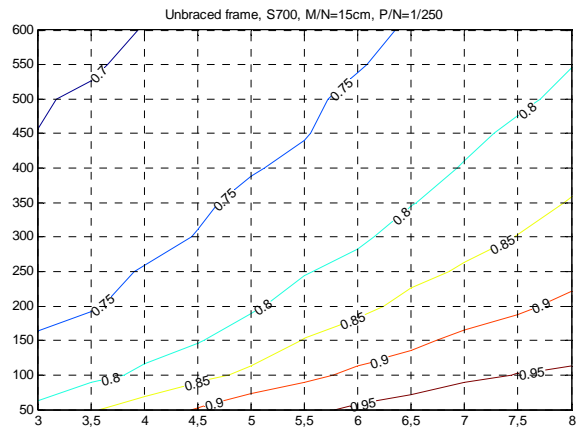


Fig.A36

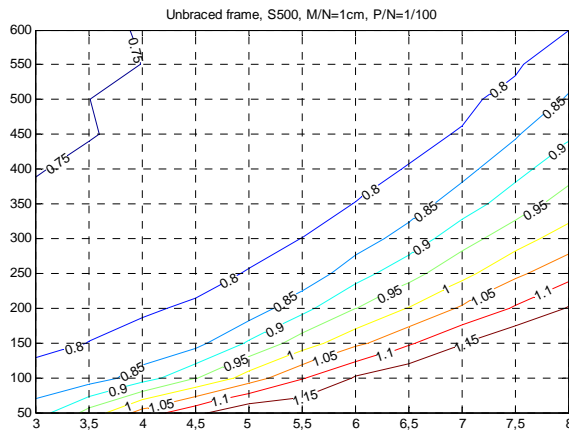


Fig.A37

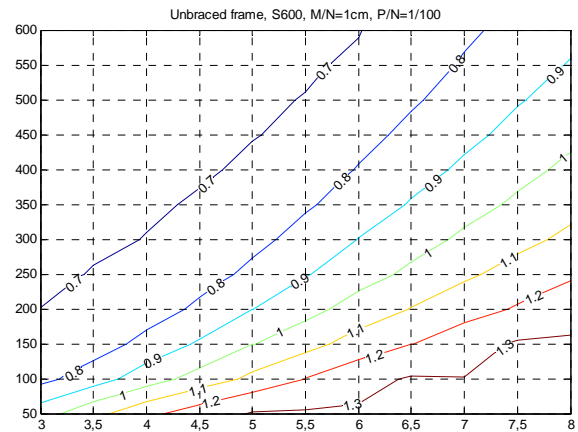


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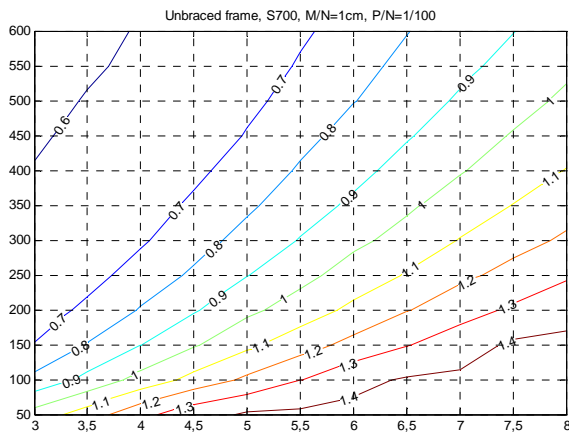


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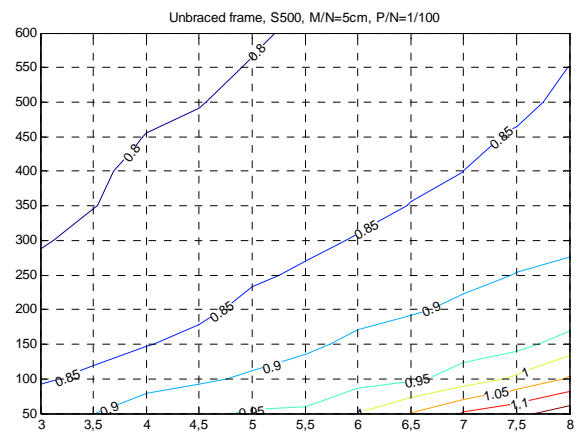


Fig.A40

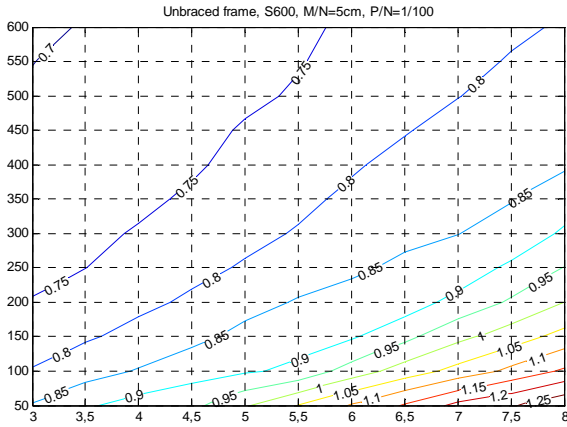


Fig.A41

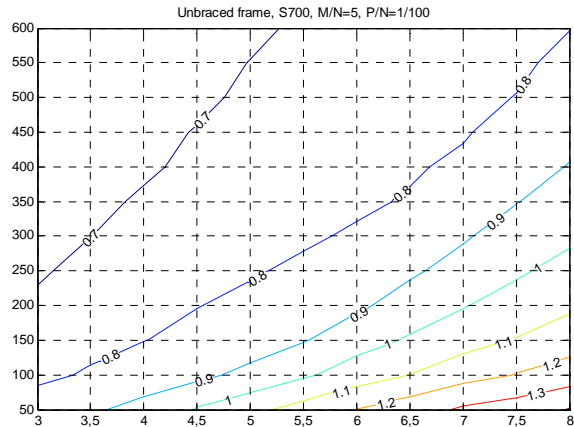


Fig.A42

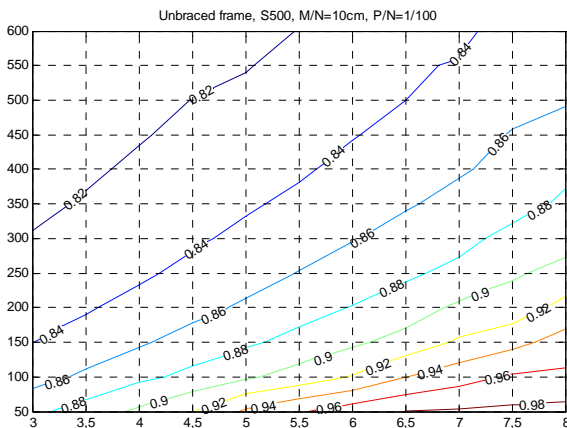


Fig.A43

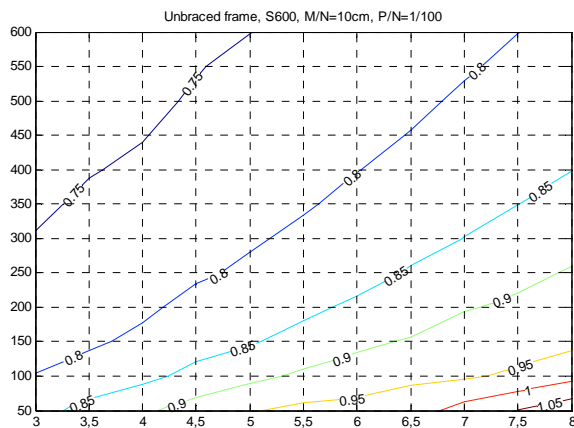


Fig.A44

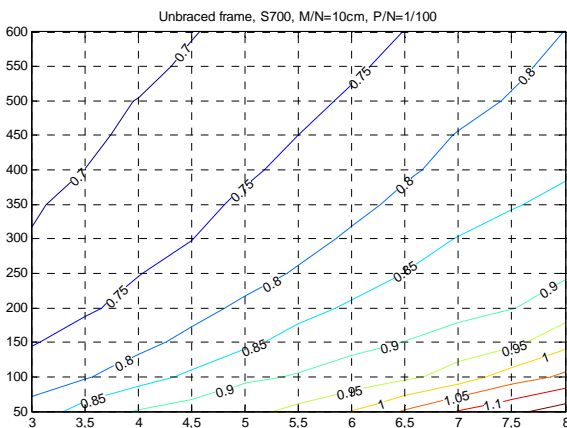


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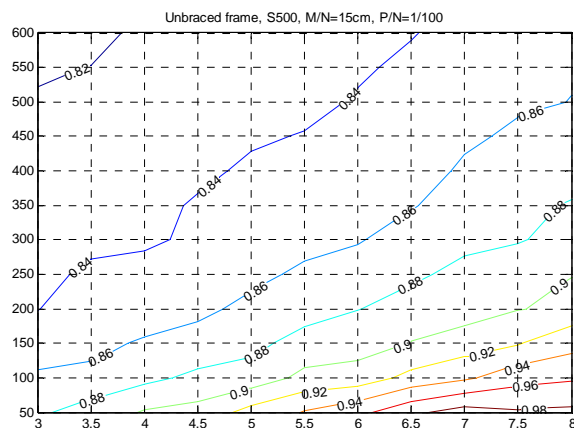


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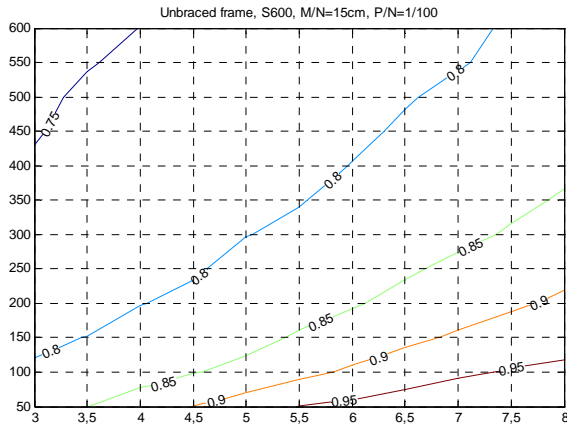


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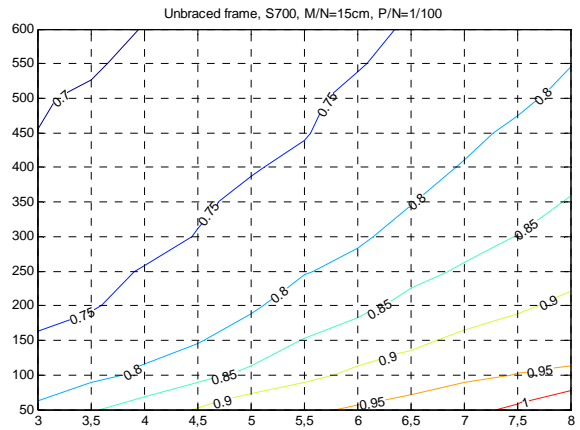


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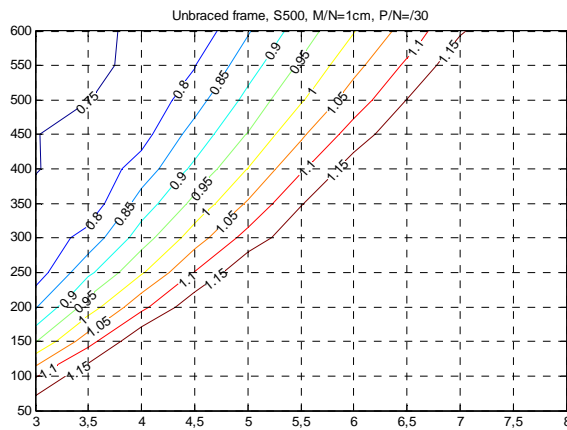


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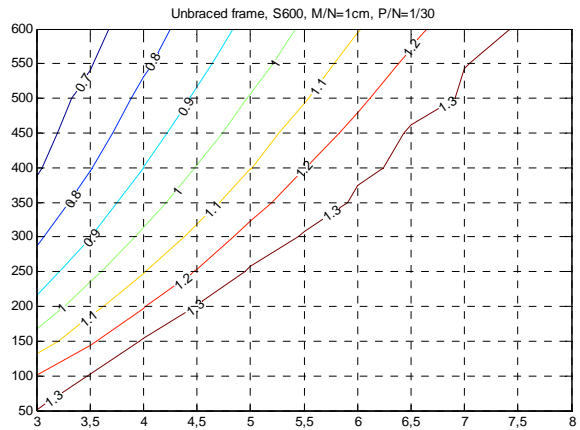


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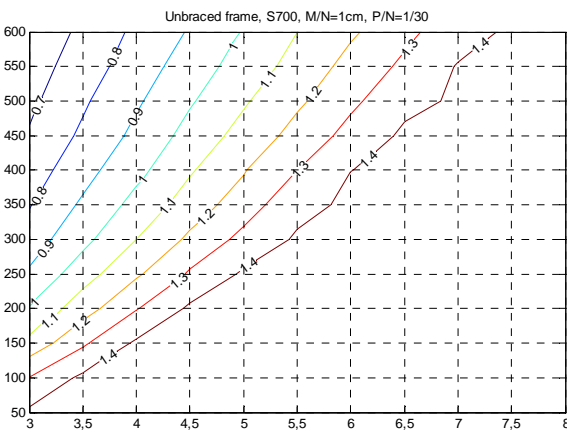


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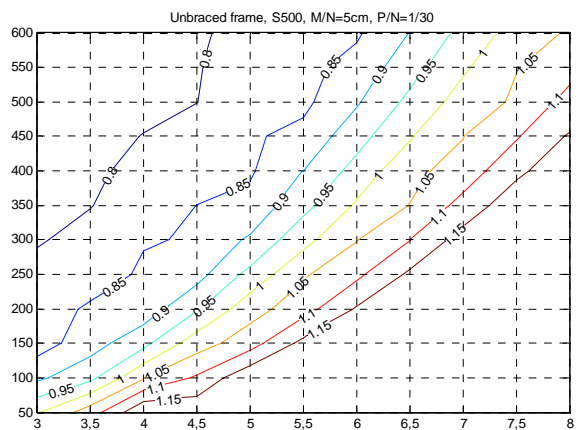


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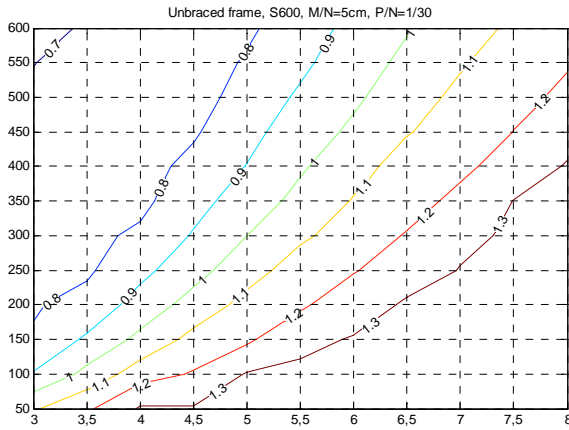


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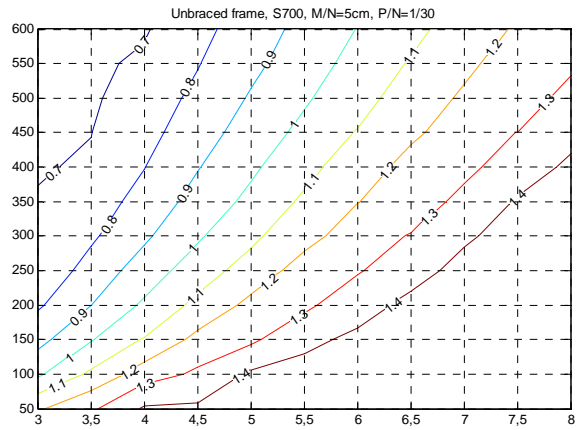


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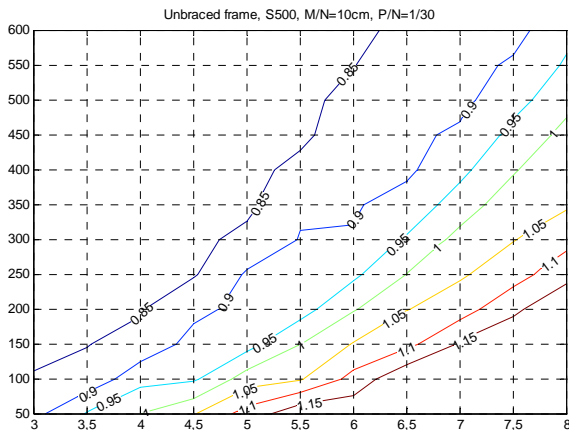


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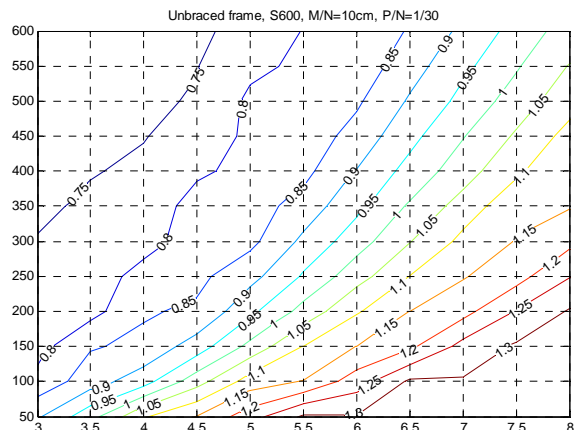


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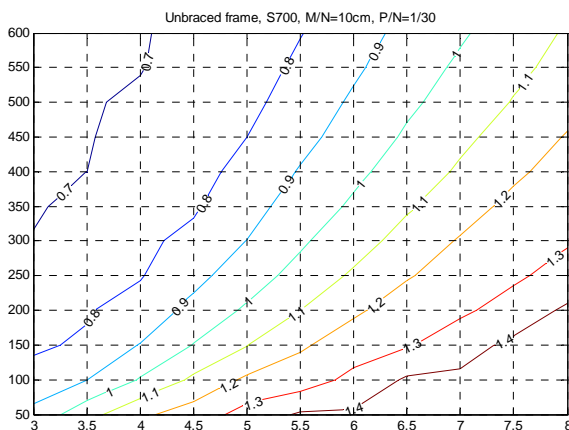


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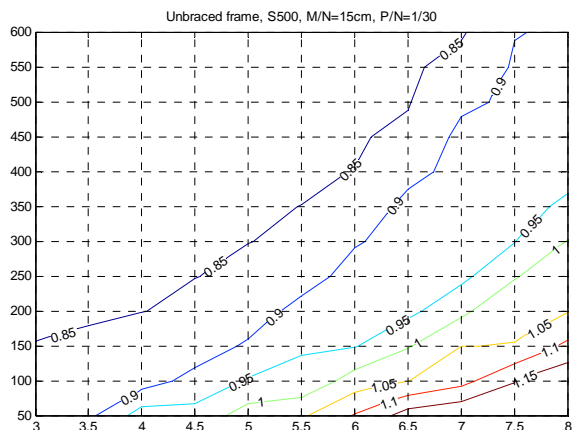


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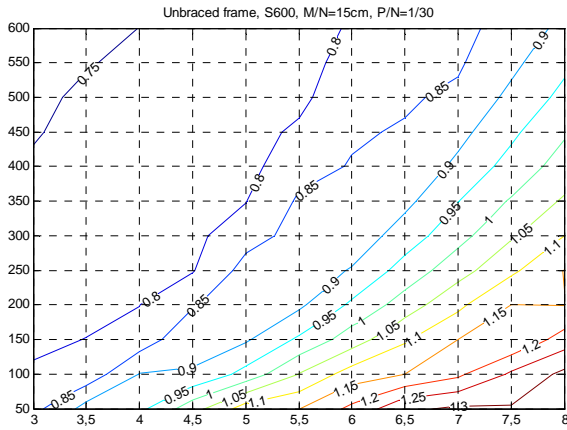


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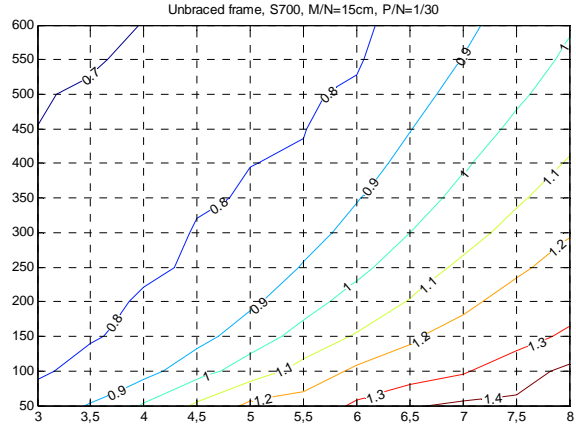


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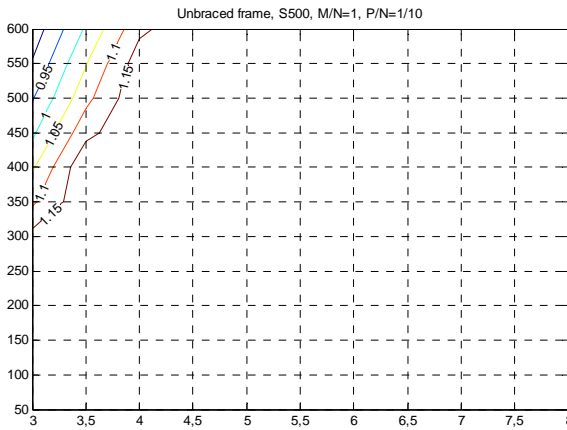


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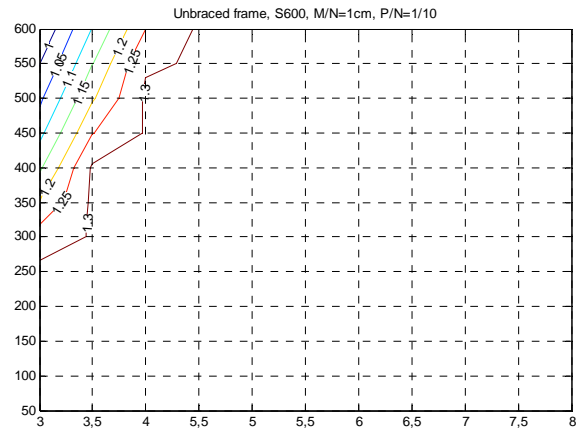


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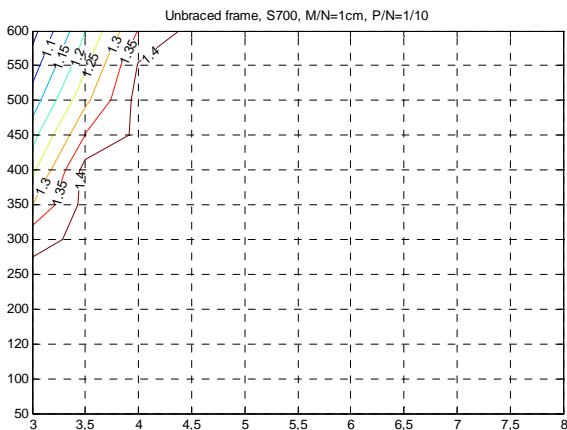


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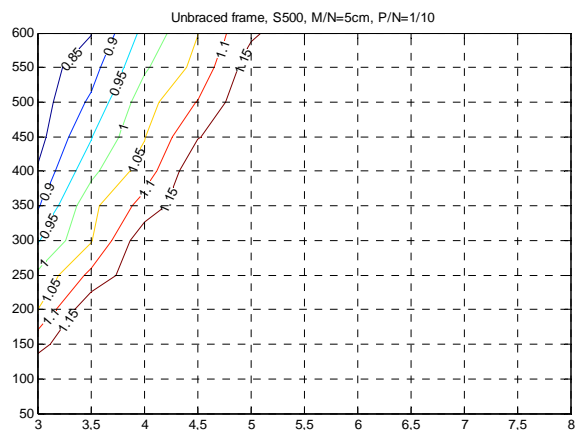


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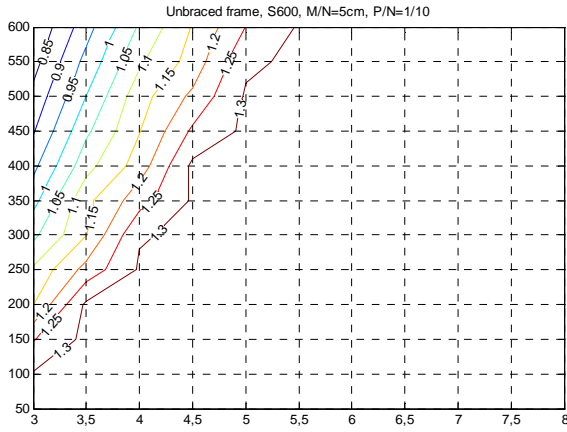


Fig.A65

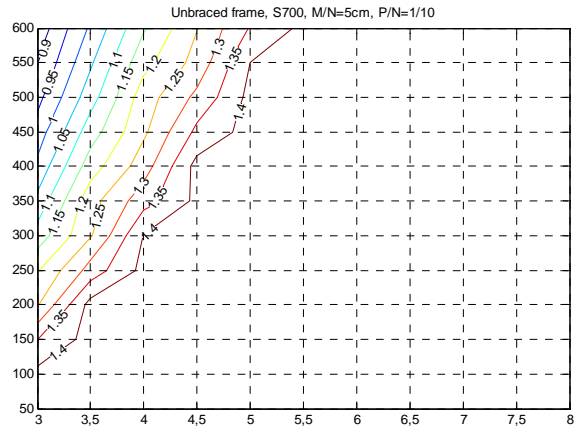


Fig.A66

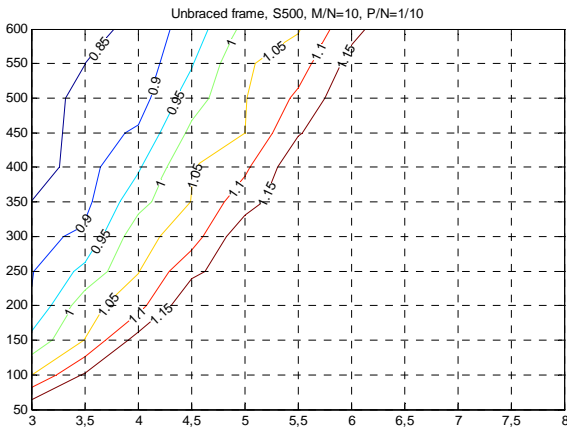


Fig.A67

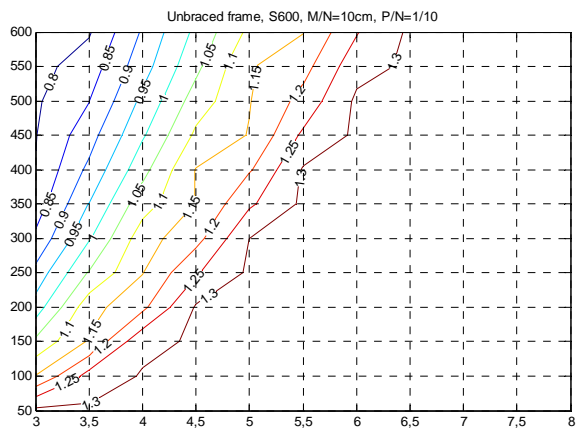


Fig.A68

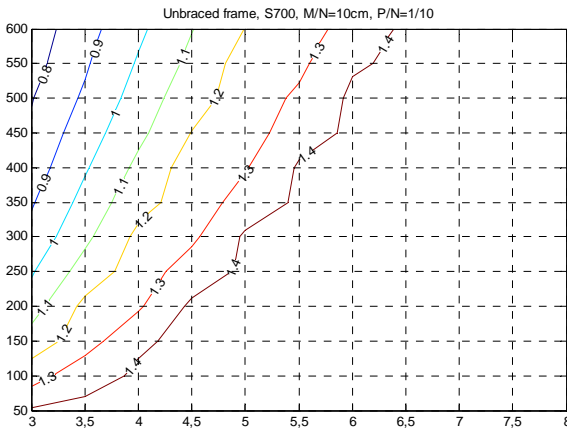


Fig.A69

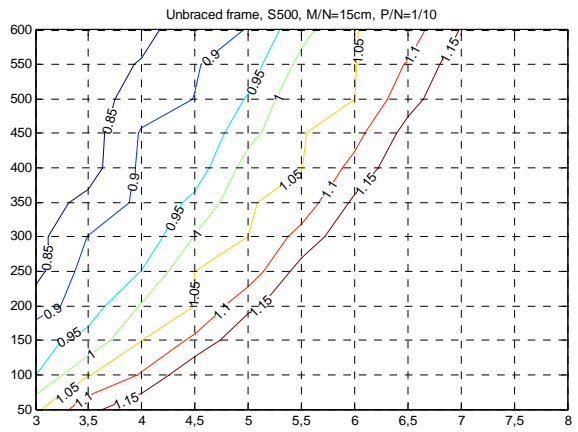


Fig.A70

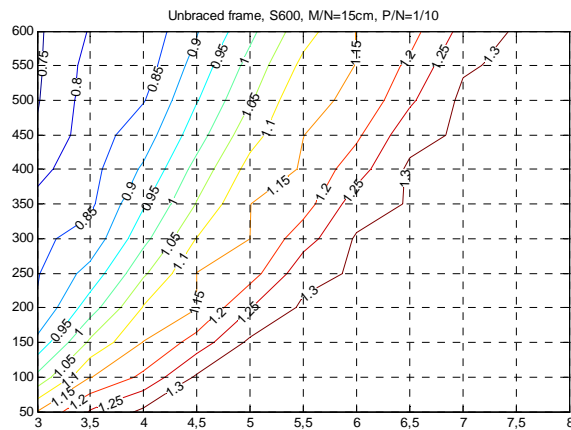


Fig.A71

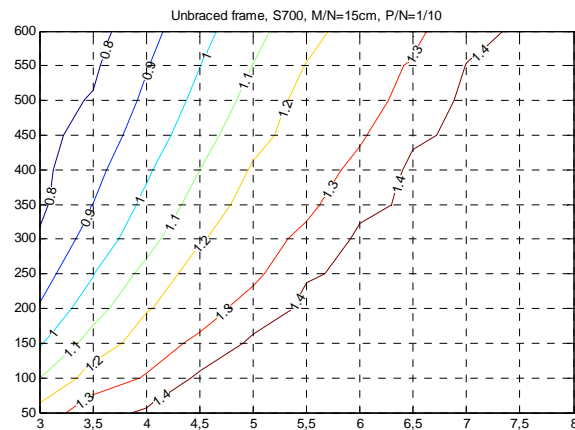


Fig.A72

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# **Appendix 2: Design of Building Type 1**

**UNIVERSITY OF LIEGE**

**Department ArGEnCO**

**MS<sup>2</sup>F Division**

# **Study case I – Building under Static and Fire loading**

***ATTEL Project, Internal report***

Hoang Van Long

Jean François Demonceau

Jean Pierre Jaspart

June 2009

# 1. Introduction

In the first period of ATTEL project, one of Liege University's tasks is to propose a reference building under static and fire loading [3]. From this building, the structural elements (columns, beam-to-column joints, column bases) will be extracted to study through the next period of the project. In the context, Liege group aims to the following purposes:

- To point out a case where the high strength steel (HSS) gives the economic interest in comparison with normal steel (S355 is considered as the reference steel).
- To point out some structural components these need to be developed in both experimental and theoretic aspects.

With the mentioned targets, the report doesn't present the detail calculations that were clearly explained in open references. Only, but enough, the necessary information and comments concerning to structural model, applied loads, analysis method, and main results are reported.

# 2. Description of the study case

As the conclusion drawn by [10], HSS only gives benefit in frames in which the columns are dominant by axial force. Therefore, a shopping centre showing of Fig.1 is taken as the ULg's study case for ATTEL project. Two values of span,  $B=8\text{m}$  and  $B=12\text{m}$  (Fig.1), are simultaneously considered.

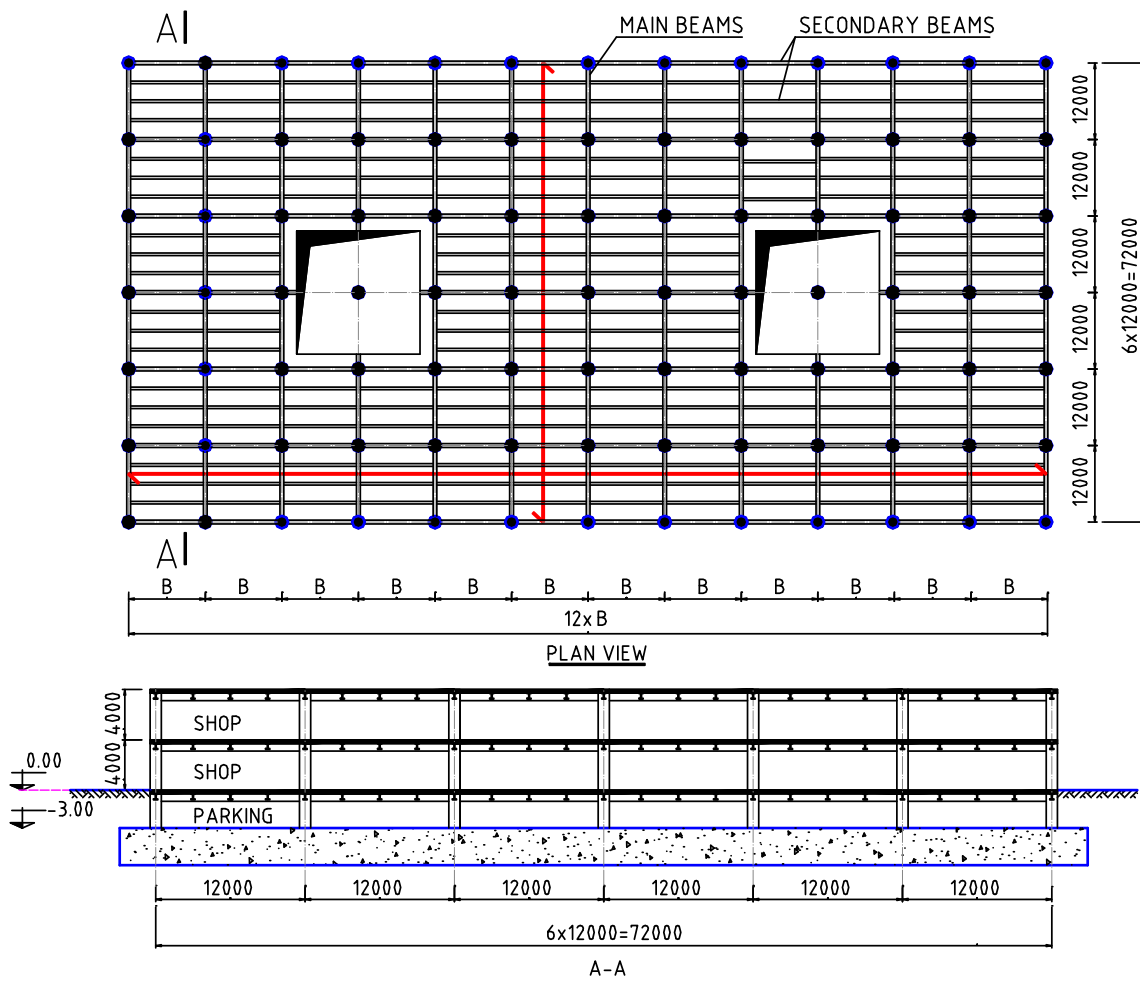


Fig.1. Plan view and main frame of the building

### 3. General solution

Reinforcement concrete slab, steel beams and shear connections constitutes composite beams (Fig.2).

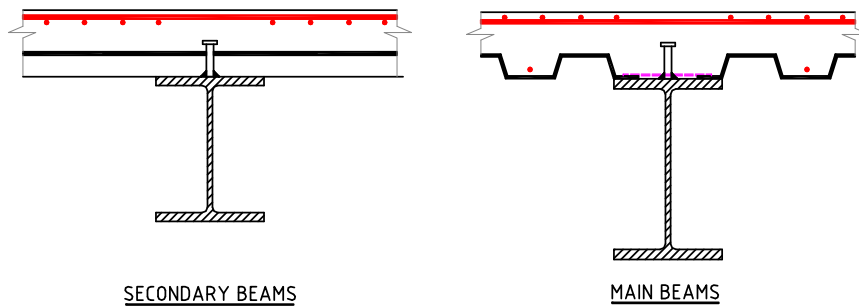


Fig.2. Composite beams

Simple beam-to-column joints are adopted (Fig.3): They are hinges in the construction phase but they are semi-rigid and/or partial strength joints in the exploitation phase. Column bases (Fig.4) are simple but their stiffness is sufficient to keep the stability of frame in the construction phase and also to receive bending moments in the exploitation phase.

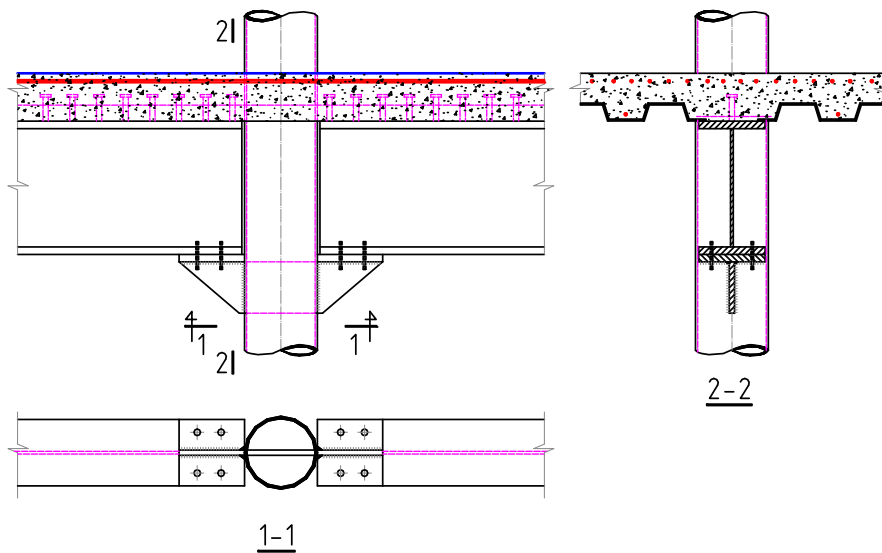


Fig.3. Beam-to-column joints

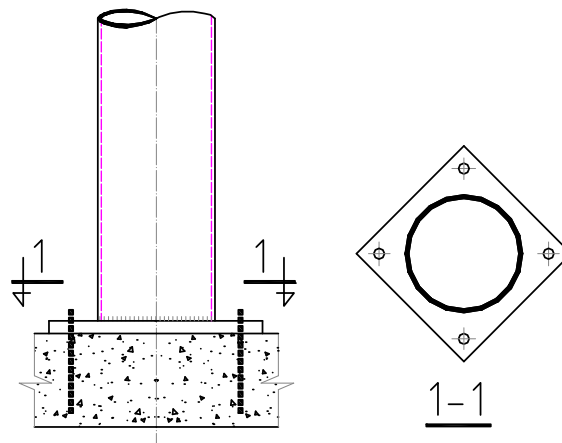


Fig.4. Column basis

In the construction phase, one doesn't use vertical supports. It means that: simple steel beams (with two hinges at the ends) must support construction loads; and global stability of frame with simple beams (beam-to-column joints are hinges) needs to be adequately examined.

The technology solutions (e.g. "contre-fleche" beams, etc) are provided to control the serviceability limit states. Therefore, the calculation concerning these ones (deformation, cracks, etc) aren't mentioned in present report.

#### 4. Static design of the building

This section presents static design of beams, columns, beam-to-column joints, and column bases in both phases: construction and exploitation. Static analysis is realized by SAP2000 program, while design of sections is carried out by internal programs based on Eurocode\_3 (steel sections) and Eurocode\_4 (composite sections).

##### 4.1. General actions

###### 4.1.1. Imposed load [4]:

Floors 1, 2 (category D):  $4,0 + 0,8 = 4,8 \text{ kN/m}^2$  (0,8 for movable partitions with a self-weight  $\leq 2,0 \text{ kN/m}$  wall length).

Top floor (roof):  $2,0 \text{ kN/m}^2$ .

###### 4.1.2. Construction load [8]: see Fig.5.

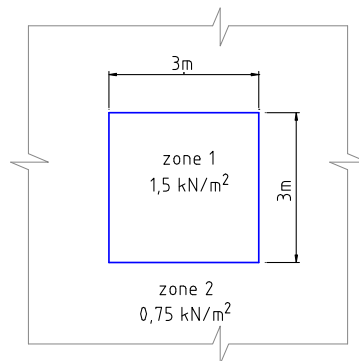


Fig.5. Construction loads

###### 4.1.3. Wind load [1, 5]:

The wind load are calculated for the height  $z = 8 \text{ m}$  (top floor). The fundamental value of the basis wind velocity equal to  $32,2 \text{ m/s}$  [6]. Suppose that the building is constructed in zone IV (classe IV). Detail calculation is presented in Table 1.

##### 4.2. Designed slab

Using the steel sheeting Cofraplus 60 furnished by ArcelorMittal (Fig.6), the dimension of slab is shown on Fig.7. The detail calculation of slab doesn't present in this report.

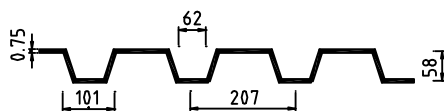


Fig.6. Steel sheeting (Cofraplus 60 [7])

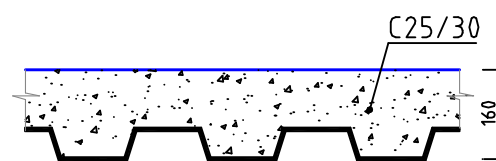


Fig.7. Slab dimension

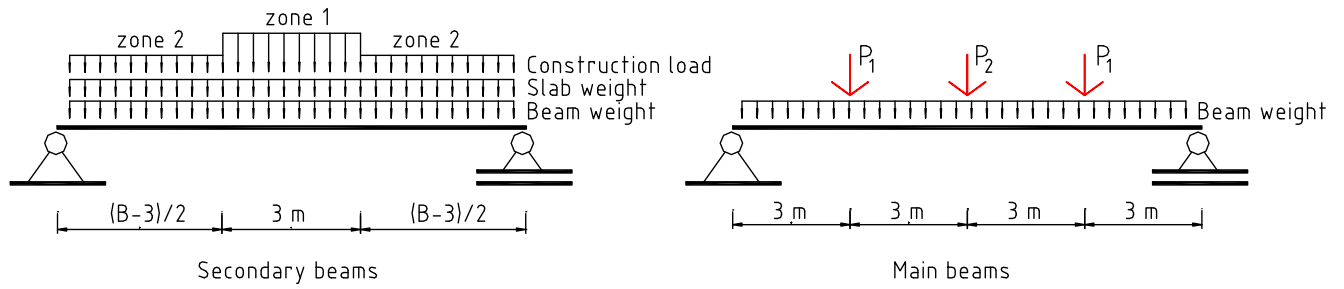
From the dimension of slab (Fig.7), we have the weight of slab equal to 3,57 kN/m<sup>2</sup>, while the weight of cover is supposed of 1,00 kN/m<sup>2</sup>.

**Table 1:** Wind load

N <sup>o</sup>	Quantities	Symbol	Expression	Value	Unit
1	Fundamental value of the basis wind velocity	V <sub>b,0</sub>	[1]	32,2	m/s
2	Air density	ρ	[5]	1,25	kG/m <sup>3</sup>
3	Direction factor	c <sub>dir</sub>	[5]	1,0	-
4	Season factor	C <sub>season</sub>	[5]	1,0	-
5	Basis wind velocity	v <sub>b</sub>	$c_{dir} C_{season} v_{b,0}$	32,2	m/s
6	Basis velocity pressure	q <sub>b</sub>	$q_b = \frac{1}{2} \rho v_b^2$	648,0	N/m <sup>2</sup>
7	Roughness length (category III)	z <sub>0,III</sub>	[5]	0,3	m
8	Roughness length (category II)	z <sub>0,II</sub>	[5]	0,05	m
9	Minimum height	z <sub>min</sub>	[5]	5,0	m
10	Maximum height	z <sub>max</sub>	[5]	200,0	m
11	Terrain factor	k <sub>r</sub>	$0.19 \left( z_{0,III} / z_{0,II} \right)^{0.07}$	0,22	-
12	Turbulence factor	k <sub>I</sub>	[5]	1,0	-
13	Orography factor	c <sub>0(z)</sub>	[5]	1,0	-
14	Standard deviation of turbulence	σ <sub>v</sub>	$k_r v_b k_I$	7,08	m/s
15	Roughness factor	c <sub>r(z)</sub>	$k_r \ln \left( \frac{z}{z_{0,II}} \right)$	1,11	-
16	Mean wind velocity	v <sub>m(z)</sub>	$c_{r(z)} c_{0(z)} v_b$	35,74	m/s
17	Turbulence intensity	I <sub>v</sub>	$\sigma_v / v_{m(z)}$	0,20	-
18	Peak velocity pressure	q <sub>p(z)</sub>	$[1 + 7 I_v(z)] q_b$	907,2	N/m <sup>2</sup>
19	Pressure coefficient (total for two faces)	c <sub>pe</sub>	[5]	1,0	-
20	Wind pressure on surface	w <sub>e</sub>	$q_{p(z)} c_{pe}$	907,2	N/m <sup>2</sup>

#### 4.3. Analysis and design of beams in the construction phase

Structural model of beams is shown on Fig.8. The remark results are presented on Table 2.



**Fig.8.** Beam modelling in the construction phase

**Table 2:** Remark values of beam's calculation in the construction phase

Parameters (See Fig.8)	Secondary beams		Main beams	
	B=8 m	B=12 m	B=8 m	B=12 m
Profile	IPE 300	IPE 400	IPE 600	IPE 750x147
Beam weight (kN/m)	0,42 (ULS & SLS) <sup>(*)</sup>	0,66 (ULS & SLS)	1,22 (ULS & SLS)	1,47 (ULS & SLS)
Slab weight (kN/m)	14,46 (ULS) 10,71 (SLS)	14,46 (ULS) 10,71 (SLS)	-	-
Construction load (kN/m)	3,38 (zone 2, ULS) 6,75 (zone 1, ULS) 2,25 (zone 2, SLS) 4,50 (zone 1, SLS)	3,38 (zone 2, ULS) 6,75 (zone 1, ULS) 2,25 (zone 2, ELS) 4,50 (zone 1, SLS)	-	-
$P_1$ (kN)	-	-	148,0 (ULS) 109,0 (SLS)	222,0(ULS) 163,4 (SLS)
$P_2$ (kN)	-	-	168,3 (ULS) 122,5 (SLS)	242,3 (ULS) 176,9 (SLS)
$M_{Ed}$ (kNm)	162,3	358,9	970,9	1419,4
$M_{Rd}$ (elastic) (kNm)	197,8	410,3	1089,5	1565,9
$f/L$ (relative deflection)	1/165	1/134	1/219	1/273

(\*) ULS = Ultimate limit states; SLS = Serviceability limit states

#### 4.4. Mechanic characteristics of beams in the exploitation phase

The stiffness of composite beams depends on effective width and sign of bending moment (due to cracks). They are resulted on Table 3.

**Table.3.** Mechanic characteristics of composite beams

Span	Beams	Effective width (cm)		Second moment (cm <sup>4</sup> )	
		Non-cracked	Cracked	Non-cracked	Cracked
B=8 m	Secondary beams	140	100	33755	14840
	Main beams	210	150	170528	94515
B=12 m	Secondary beams	210	150	76636	36585
	Main beams	210	150	308679	166607

#### 4.5. Modelling of joints

For the global analysis, beam-to-column joints and column bases are modelled as the show of Fig.9, the values in which will be explained in Section 4.10.

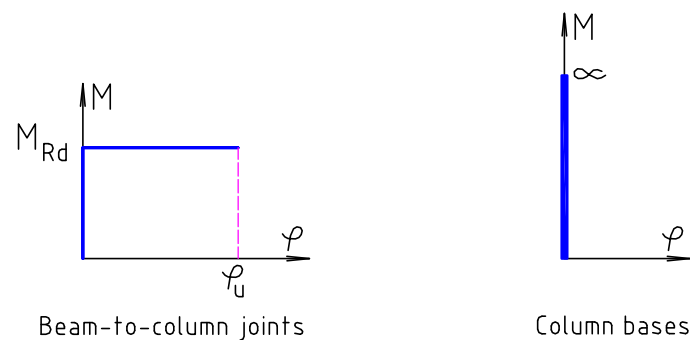


Fig.9. Modelling of joints

#### 4.6. Static analysis and design of secondary beams

In the exploitation phase, secondary beams are considered as the continuous beams with variable stiffness (Table 3). The beams cut by the technique hole (Fig.1.) are the unfavourable, and they are chosen to design. Modelling of the beams is reported on Fig.10, while the results are summarized in Table 4.

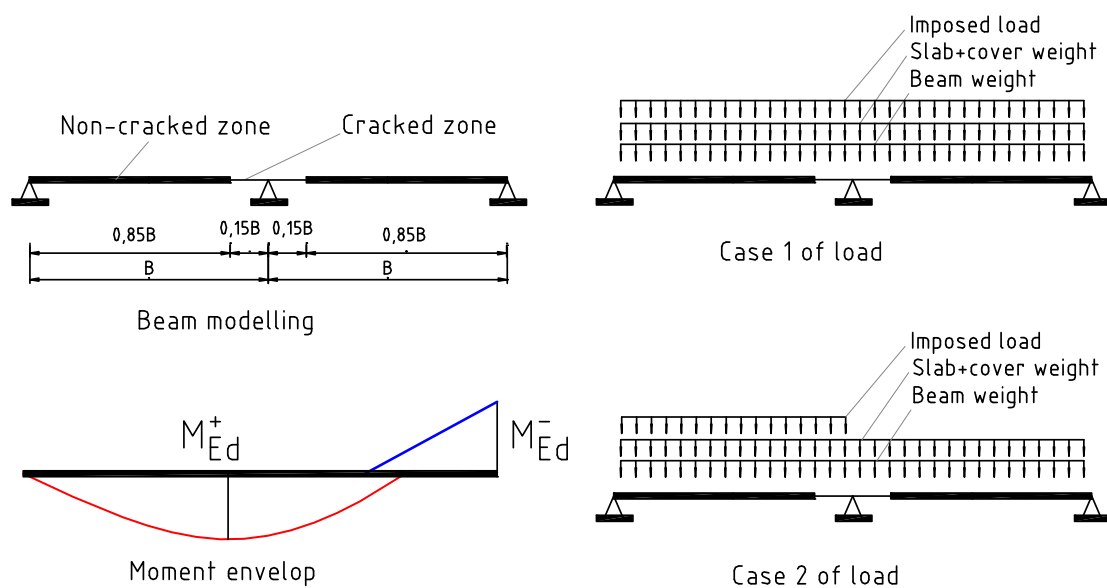


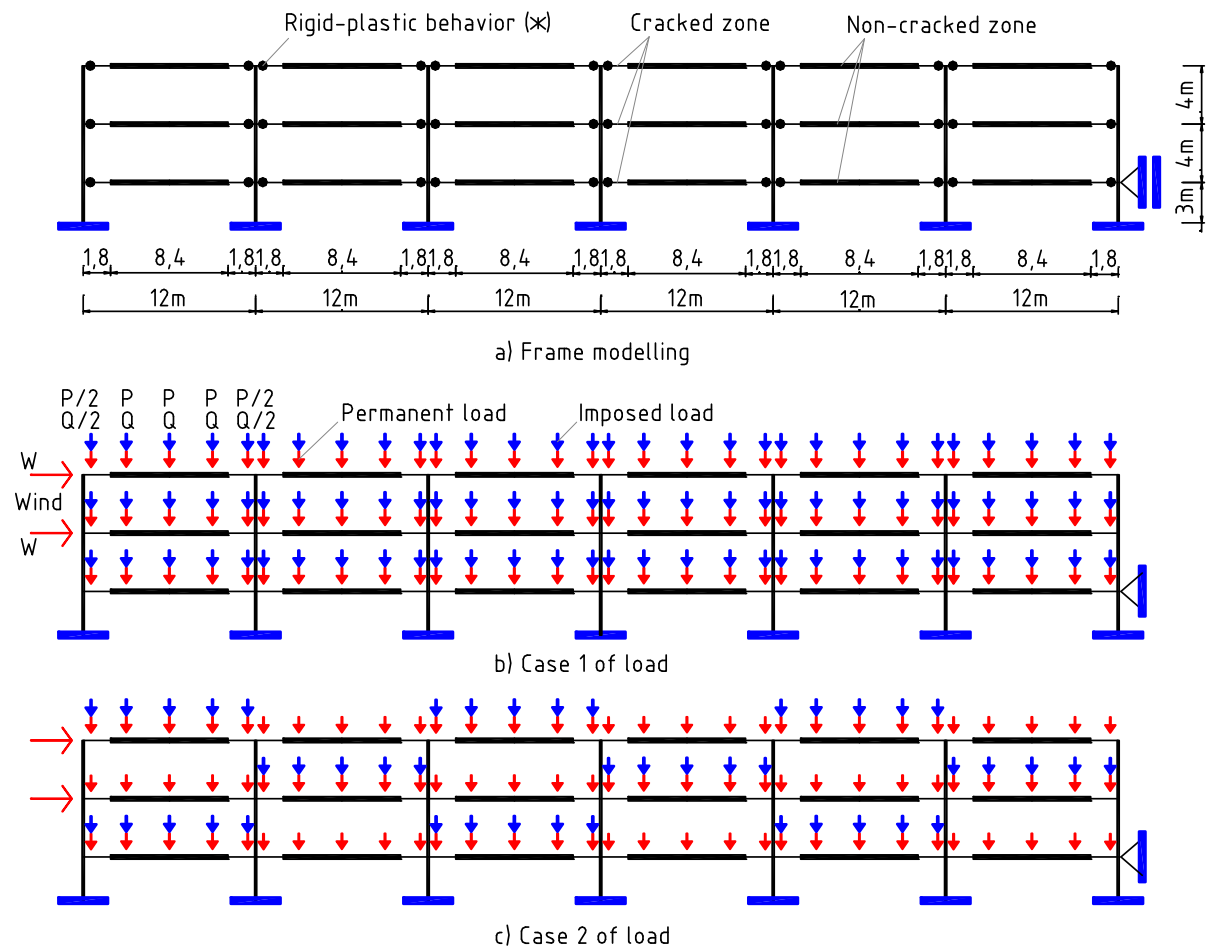
Fig.10. Static analysis of secondary beam (exploitation phase)

**Table 4:** Results of secondary beam design

Parameters	B=8 m	B=12 m
Beam weight (kN/m)	0,42	0,66
Slab + cover weight (kN/m)	18,58	18,58
Imposed load (kN/m)	21,60	21,60
$M_{Ed}^+$ (kNm) <sup>(*)</sup>	240,00	537,0
$M_{Ed}^-$ (kNm) <sup>(*)</sup>	186,00	430,00
$M_{Rd}^+$ (kNm)	530,9	926,9
$M_{Rd}^-$ (see Table 12) (kNm)	378,00	463,74

Remark: (\*) these values were redistributed 25%.

#### 4.7. Static analysis and design of frame in the exploitation phase



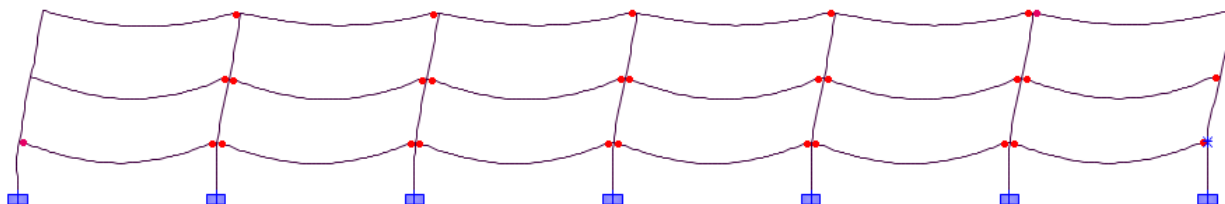
**Fig.11.** Modelling of main frames, ((\*) see Fig.9)

The plastic capacity of middle section of main beams is very greater than which of joints. Therefore, using the linear analysis and taking the redistribution of moments aren't suitable. Moreover, the global stability of frame is reduced due to the use of HSS that leads to the second effect is maybe significant. The plastic-hinge analysis including P- $\Delta$  effect is so chosen. The rigid-plastic behaviour (Fig.9) is assigned to beam-to-column joints (Fig.11). In prior, the tubes 350x12 are assigned to columns. Applied loads are presented in Table 5.

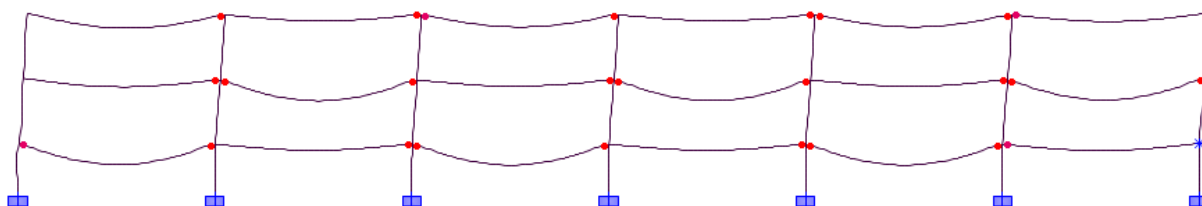
Some results of frame analysis are given on Figs. 12-19, and Table 6. Using internal forces in Table 6 and optimization solution [10] we have the optimal sections (Table 7) for columns with two steel grades S355 and S690.

**Table.5.** Frame's loads

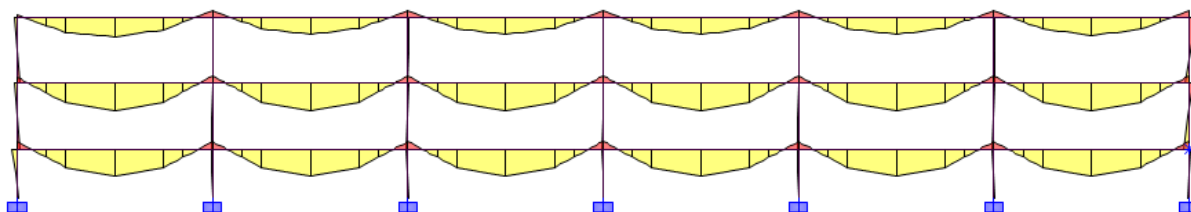
Level	Load (Fig.11)	Span	
		B=8 m	B=12 m
Floor	P (kN)	205,6	310,5
	Q (kN)	184,7	278,9
Roof	P (kN)	85,7	129,4
	Q (kN)	184,7	278,9
Floor/roof	W (kN)	46,5	69,8



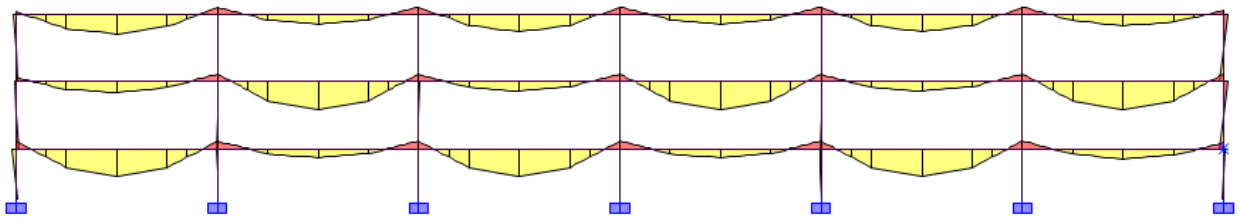
**Fig.12.** Deformation and distribution of plastic hinges (B=12m, case 1 of load)



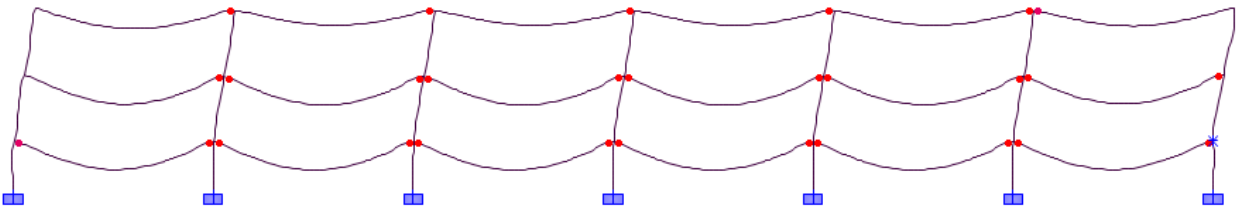
**Fig.13.** Deformation and distribution of plastic hinges (B=12m, case 2 of load)



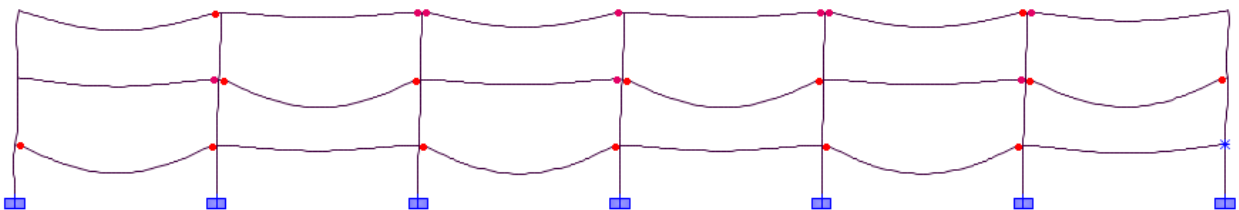
**Fig.14.** Distribution of bending moment (B=12m, case 1 of load)



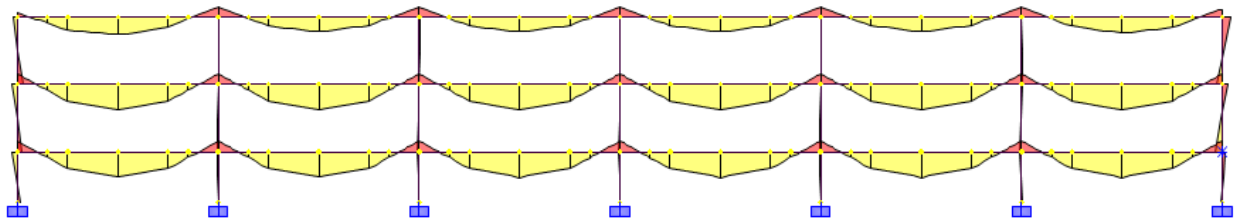
**Fig.15.** Distribution of bending moment (B=12m, case 2 of load)



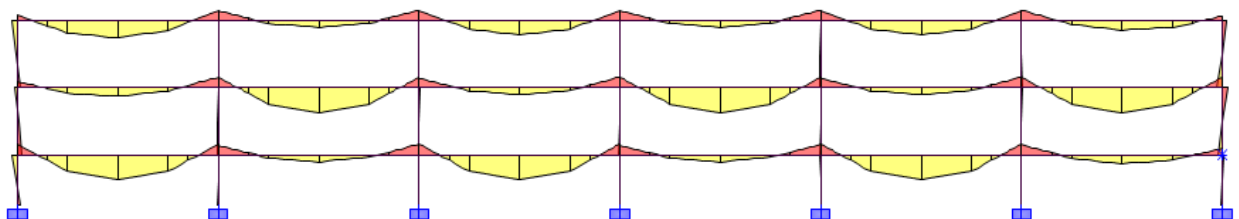
**Fig.16.** Deformation and distribution of plastic hinges (B=8m, case 1 of load)



**Fig.17.** Deformation and distribution of plastic hinges (B=8m, case 2 of load)



**Fig.18.** Distribution of bending moment (B=8 m, case 1 of load)



**Fig.19.** Distribution of bending moment (B=8 m, case 2 of load)

**Table 6:** Remarks results of frame analysis

Span	Plastic-hinge		Sagging moment in beams	Internal forces in columns (chosen values to be design)						
				First combination			Second combination			
				$\varphi_u$ (mrad)	$\varphi_{Ed\ max}$ (mrad)	$M_{Rd}$ (kNm)	$M_{Ed}$ (kNm)	$M_{Ed}$ (kNm)	$N_{Ed}$ (kN)	$Q_{Ed}$ (kN)
B=8 m	25	17	1967	1532	78,3	3986,0	32,2	353,0	1506,0	173,0
B=12 m	25	24	2789	2779	127,4	6397,0	48,7	464,7	2481,0	234,2

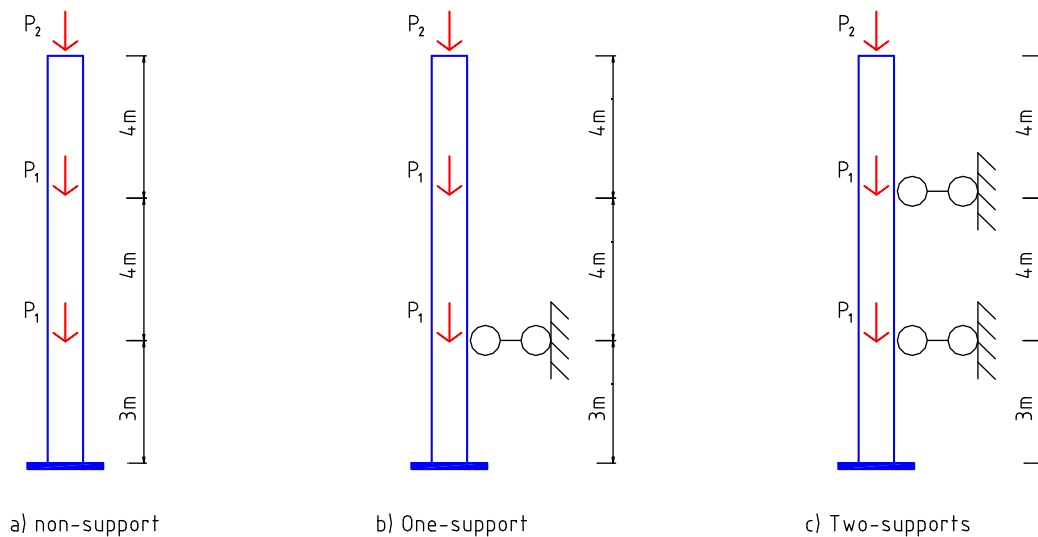
**Table.7.** Design of columns and economic interest of HSS

Span	S355		S690		Column-S690' cost/ Column-S690' cost with			
	Diameter (cm)	Thickness (cm)	Diameter (cm)	Thickness (cm)	c=1,1 <sup>(*)</sup>	c=1,2	c=1,3	c=1,4
B=8 m	50,14	0,84	28,64	0,93	0,68	0,75	0,81	0,87
B=12 m	62,35	1,05	35,0	1,14	0,66	0,72	0,78	0,84

(\*): c is the cost rate (cost of S690 tube's weight unity / cost of S355 tube's weight unity)

#### 4.8. Verification of columns in the construction phase

Because the beam-to-column joints are hinges in the construction phase, the structural model for columns is the show of Fig.20. The horizontal supports are provided if it is necessary. The dimensions of column's sections are mentioned in Table 7. The applied loads and the buckling load factors are shown in Tables 8 and 9 respectively.



**Fig.20.** Structural model of columns in the construction phase

**Table 8:** Applied load in columns in the construction phase

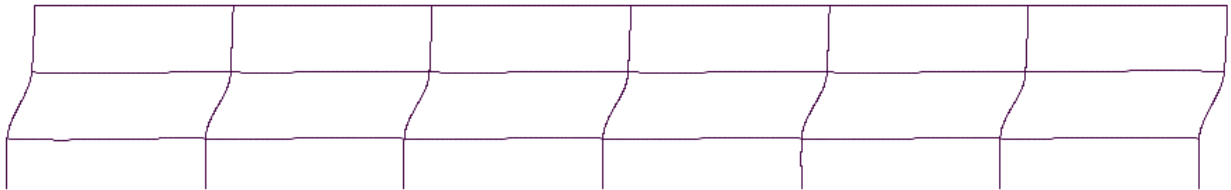
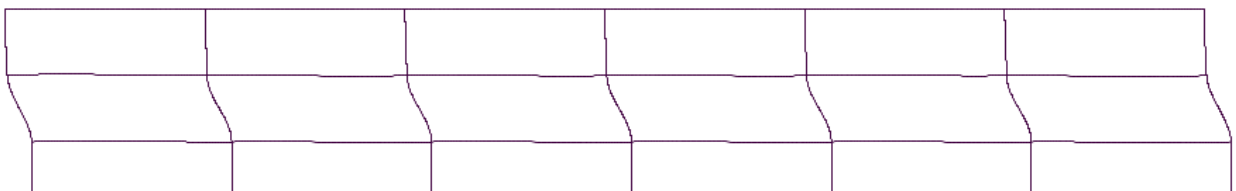
Span	Load (design values)	
	$P_1$ (kN)	$P_2$ (kN)
B=8 m	494,11	612,24
B=12 m	751,25	923,38

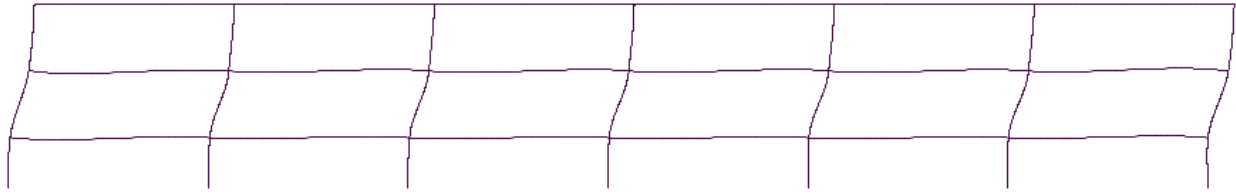
**Table 9:** Load factor leading to buckling of columns in the construction phase

Span	S355 columns (see Table)			S690 columns (see Table)		
	No support	One support	Two supports	No support	One support	Two supports
B=8 m	2,00	3,43	11,44	0,40	0,69	2,32
B=12 m	3,21	5,35	17,80	0,59	1,02	3,41

#### 4.9. Global stability of frames

Figs. 21-24 show instability forms of frames, while Table 10 present load factor that lead to global instability of frames. In these realizations, case 2 of load (Fig.11) are adopted. It indicates that global stiffness of frames decreases considerably when HSS is used.

**Fig.21.** Form of global instability of frame (B=12m, S690)**Fig.22.** Form of global instability of frame (B=12m, S355)**Fig.23.** Form of global instability of frame (B=8m, S690)



**Fig.24.** Form of global instability of frame (B=8m, S355)

**Table 10:** Load factor of global instability of frames

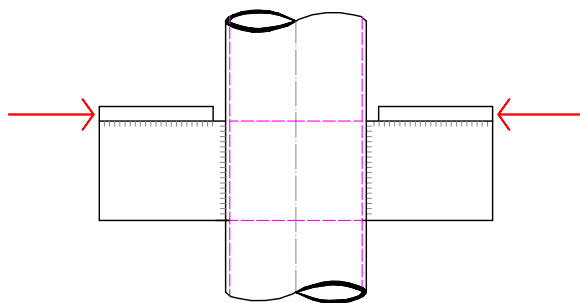
Span	S355 columns	S690 columns
B=8 m	15,47 (non-sway frame)	4,16 (sway frame)
B=12 m	19,50 (non-sway frame)	5,50 (sway frame)

#### 4.10. Beam-to-column joints

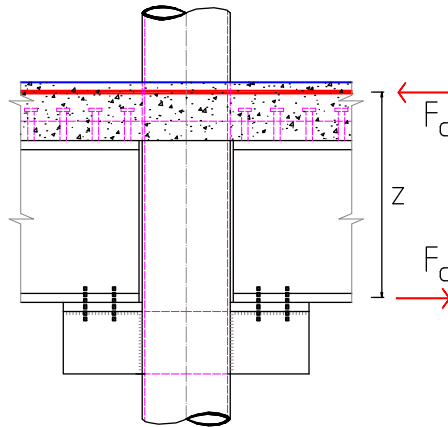
On the point of view of the component method, the used beam-to-column joint may be divided into five components as show on Table 11.

**Table 11:** List of components of beam-to-column joint

N <sup>o</sup>	Component	Remarks
1	Column piece in shear	-
2	Longitudinal slab reinforcement in tension	The ductility is needed to be studied
3	Bolts in shear	See [7]
4	Bolts in bearing	See [7]
5	Vertical plate and column in compression (Complex component):	This complex component needs to be studied in ATTEL project.



In the waiting the treatment of the fifth component (Table 11), the rigid-plastic behavior of beam-to-column joints is adopted (Fig.9). The rotation capacities of joints are fixed a value of 25 mrad (see [10]), while the moment capacities are simply calculated by the multiplier of  $F_d$  with lever arm  $z$  (Fig.25). Force  $F_d$  is the result of tension capacities of reinforcement bars in the effective width. Table 12 presents plastic capacities of joints. They are used in the frame analysis (Section 4.7)



**Fig.25.** Plastic capacity of beam-to-column joints

**Table 12:** Plastic capacities of joints

Span	Beams	Steel profiles	Reinforcement (cm <sup>2</sup> )	Lever arm z (cm)	M <sub>Rd</sub> (kNm)
B=8 m	Secondary beams	IPE300	20	43,47	378,00
	Main beams	IPE600	20	73,05	635,22
B=12 m	Secondary beams	IPE400	20	53,33	463,74
	Main beams	IPE750x143	20	88,45	769,13

#### 4.11. Column base

##### 4.11.1. Theoretic aspect

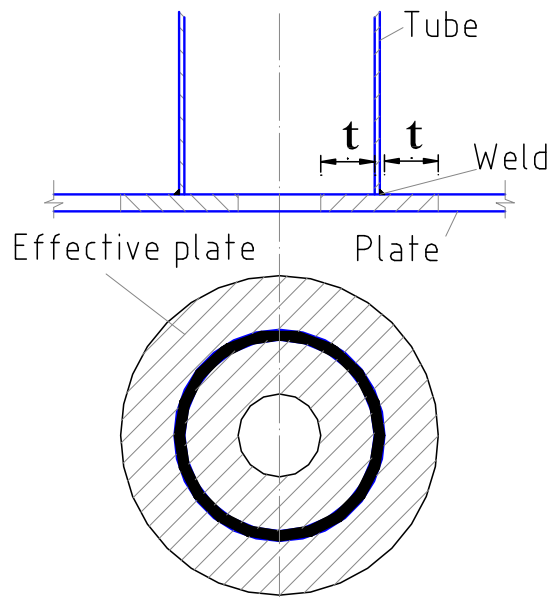
According to the configuration of column bases showing on Fig.4, the following components need to be considered (Table 13)

**Table 13:** List of components of column bases

N°	Component	Remarks
1	Section of tube (column)	It was clarified
2	Tube-to-plate weld	It was clarified
3	Anchor bolts in tension	See, for example, [9]
4	Concrete block in compression	See, for example, [9]
5	Plate in bending	Due to the shape of column (circular), this component needs to be studied, see Section 4.11.3

##### 4.11.2. Concrete block in compression

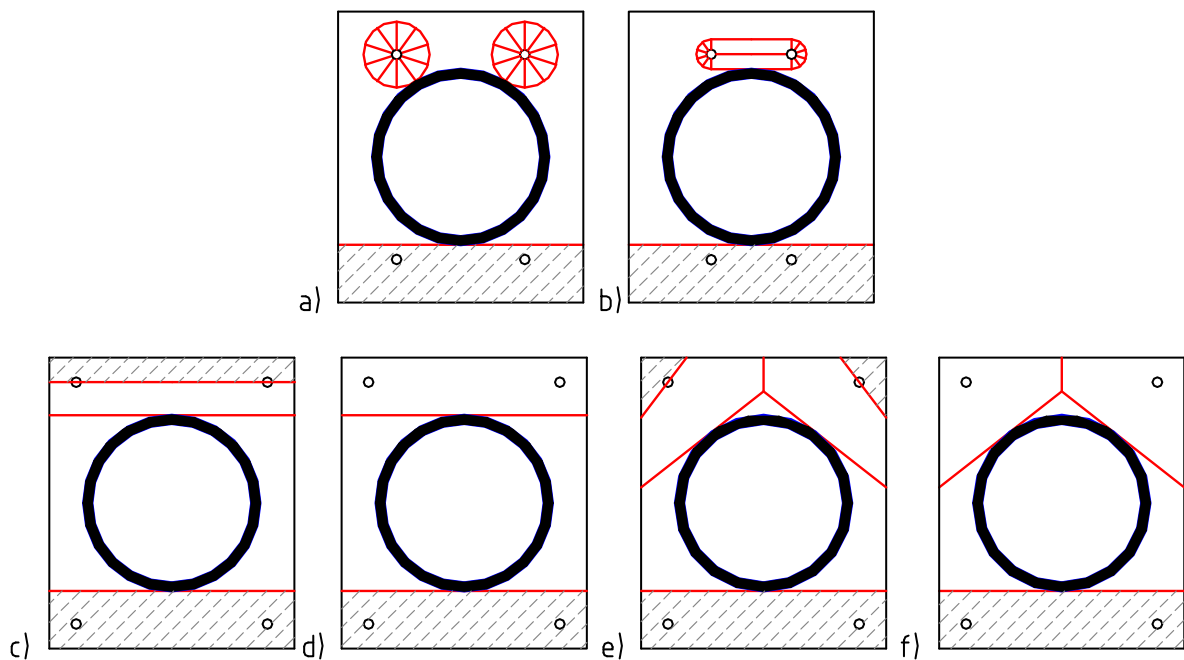
This component is presented in some references, e.g. [9]. The effective area of the concrete block in compression is as a ring (Fig.26) whit the parameter t (see [9], expression 3.4)



**Fig.26.** Effective plate

#### 4.11.3. Plate in bending

Based on the rigid-plastic analysis of plate-bolts system, one may obtain the plastic capacity of plate component. According to the relative stiffness between the plate and the bolts, and also the location of bolts, many kinds of plastic mechanics may be happened (Fig.27).

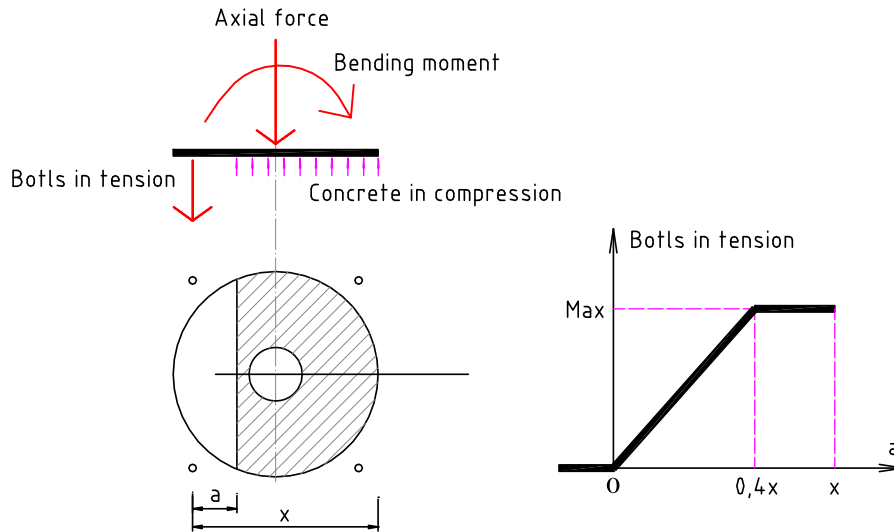


**Fig.27.** Yield line modes of failure

The local mechanics (Figs.27a and 27b) are clarified and presented in Eurocode [7] while the global mechanics (Figs.27c-27f) are rarely considered in literatures. Therefore, in present case (hollow circular columns), the plate component needs to be developed. For instant, we were concretely formulated for the failure modes shown on Figs.27e and 27f. However, they are only simple yield lines, more complicated mechanics maybe occurs in practices. A complete study of the case will be presented in next report.

#### 4.11.4. Assembly of components

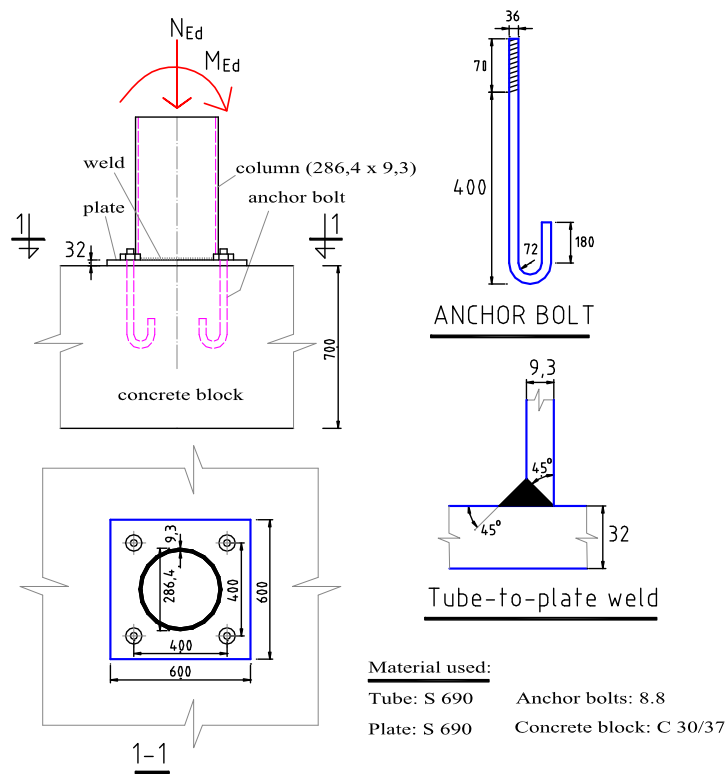
The global behaviour of plate in bending, anchor bolts in tension and concrete block in compression may be modelled as the show of Fig.28 [9]. Realizing two equilibrium equations, one obtains the values of bending moment and axial force.



**Fig.28.** Assembly of plate, anchor bolts and concrete block

#### 4.11.5. Application

The mentioned theory is applied to design the column base for the case the span  $B = 8\text{m}$ . The applied loads are shown on Table 6. The dimension of the column base is reported on Fig.29. The results are presented on Fig.30.



**Fig.29.** Dimensions of column base

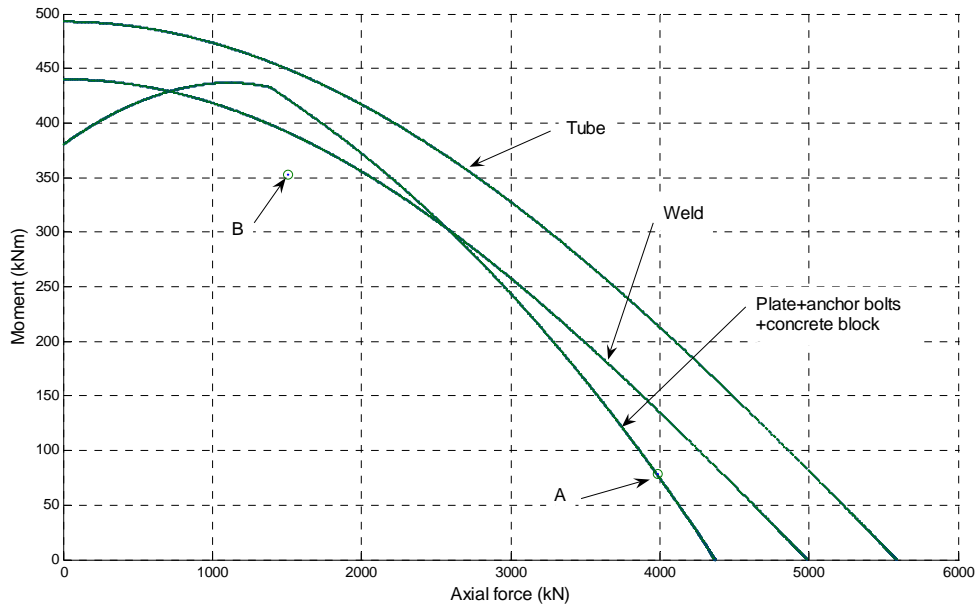


Fig.30. Interaction curve of column base (A and B are the points of applied load – see Table 6)

## 5. Building on fire condition

Clearly, steel tube columns are very weak on the fire condition. A protected solution needs to be adequately reflected so that the use of steel column are still economic than composite columns. Conversely, HSS haven't got any reasons to be used. Detail research on the last content and also on the joint behaviour on fire will be presented in next reports.

## 6. Conclusion and perspectives

Through the study case, some final remarks may be drawn as the following:

- In comparison with normal steel (S355), HSS gives interest in the proposed building (see Table 7).
- Using simple joints without vertical supports in the construction phase is a reasonable solution. It is simple to build-up, the capacity of elements are exploited in both construction and exploitation phases. The total cost is certainly decreased. However, this solution normally requires careful design (e.g. second-order plastic-hinge analysis for frames).
- On the theoretic aspect, the study case mentions some structural components that need to be developed, e.g. vertical plate in beam-to-column joints, base plate in column bases. These are the aims of next works in the framework of ATTEL project.

## References

- [1] Actions du vent sur les constructions: Generalites - Pression du vent sur une paroi et effets d'ensemble du vent sur une construction. NBN B 03-002-1, Institut Belge de Normalisation, 1988.
- [2] Anderson D (ed). COST C1 - Composite steel-concrete joints in frames for buildings: Design provisions. Brussels – Luxembourg 1999.

- [3] ATTEL project: Performance-based approaches for high strength steel tubular columns and connections under earthquake and fire loading. Six-monthly Report 1, 2008.
- [4] Eurocode 1: Actions on structures – Part 1-1: General actions- Densities, self-weight, imposed loads for buildings. EN 1991-1-1, Brussels, 2002.
- [5] Eurocode 1: Actions on structures - General actions - Part 1-4: Wind actions. EN 1991-1-4, Brussels, 2004.
- [6] Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings. EN 1993-1-1, Brussels, 2005.
- [7] Eurocode 3: Design of steel structures - Part 1-8: Design of joints. EN 1993-1-8, Brussels, 2003.
- [8] Eurocode 4: Design of composite steel and concrete structures - Part 1-1: General rules and rules for buildings. EN 1994-1-1, Brussels, 2004.
- [9] Guisse S, Vandegans D, Jaspart JP. Application of the component method to column bases – experimentation and development of a mechanical model for characterization. Research Centre of the Belgian Metalworking Industry, 1996.
- [10] Hoang VL et al. Field of application of high strength steel circular tubes for steel and composite columns from an economic point of view. A paper to be published.
- [11]. Nethercot DA. Semirigid joint action and the design of nonsway composite frames. Engineering Structures 1995 (17): 554-567.

# **Appendix 3: Design of Building Type 2**

**ATTEL PROJECT**

**PERFORMANCE-BASED APPROACHES FOR HIGH STRENGTH TUBULAR COLUMNS AND CONNECTIONS  
UNDER EARTHQUAKE AND FIRE LOADINGS**

# **UNITN study case – Building Type 2 (“Medium” earthquake + fire)**

**Internal report – June 2009**

**Authors:**

Prof. Oreste S. Bursi

Ph.D. Eng. Fabio Ferrario

Eng. Gabriele Zanon

# I Introduction

One of Trento University's tasks is to propose the design of a reference building under static, seismic and fire loading [3]. The design's objectives are the following:

- To point out a structural typology where the use of high strength steel (HSS) gives the economic interest in comparison to the use of normal steel (S355 is considered as the reference steel while S590 is considered the HSS).
- To point out some structural components that will be developed in both experimental and theoretical aspects.

This report doesn't present the detailed calculations, that were clearly explained in open references, but only the necessary information and comments concerning the structural model, the applied loads, the method of analysis and main design results.

# II Description of the study case

The reference building is a composite steel and concrete structure for offices and meeting or expositions: in fact it's provided of a central open space as shown in Figg.1, 2, 3, 4. It is designed with different types of circular columns (**CHS** = Circular hollow section and **CFT** = Concrete filled tube) of different steel grade under static, seismic and fire loading. (**S355** steel would be compare with **S590** steel).

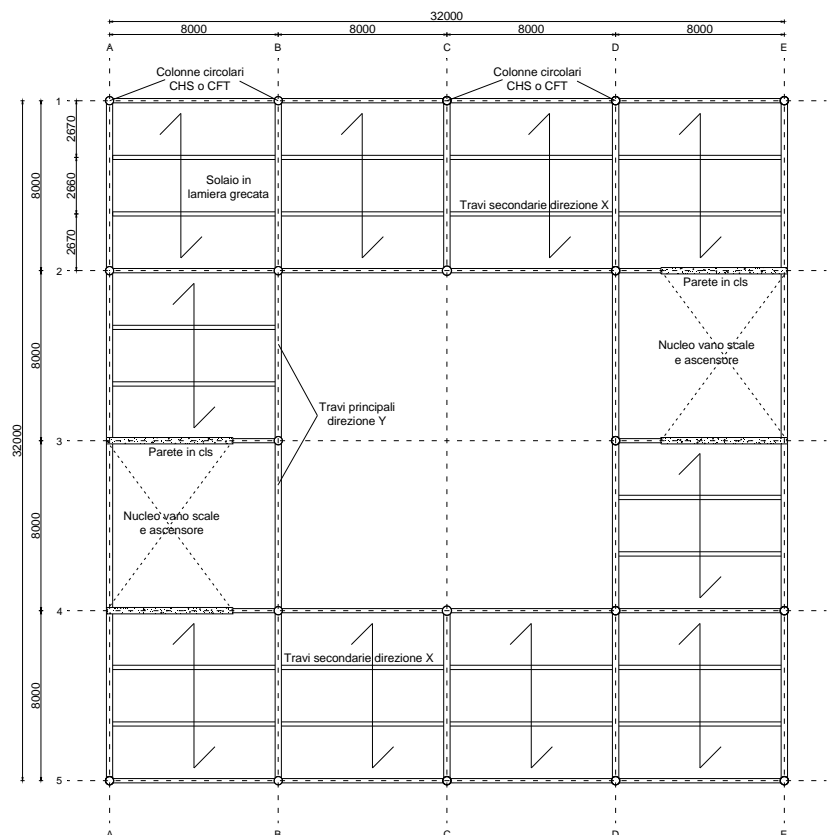


Figure 1. Plan view of the building.

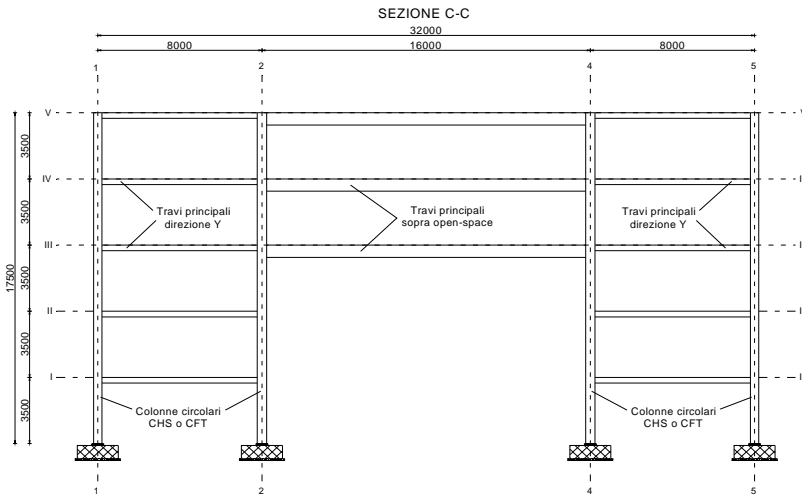


Figure 2. Main frame of the building, C-C section.

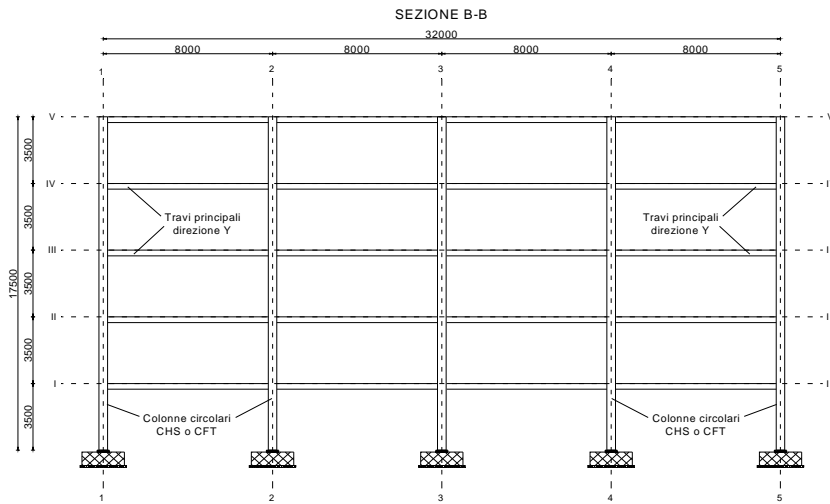


Figure 3. Main frame of the building, B-B section.

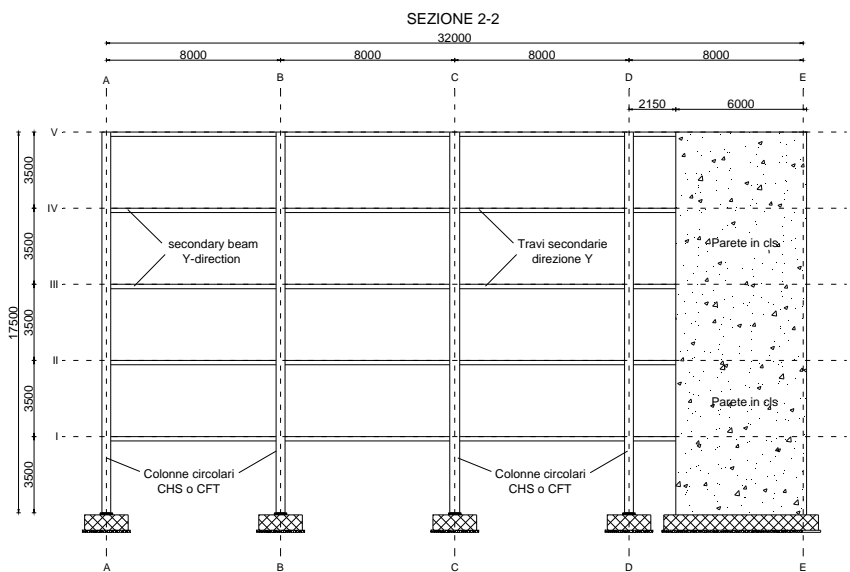


Figure 4. Main frame of the building, 2-2 direction.

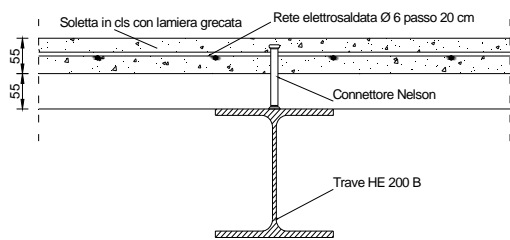
# Main elements of the building

The materials characteristics of the main elements are reported in the following Table.

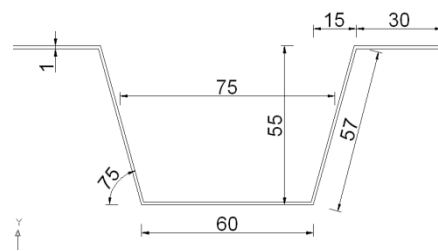
Element	Material
Concrete	C30/37
Main beams e secondary beams	S275
Columns	S355 or S590
Plates and stiffeners of the joints	S355
Nelson' s studs	S450
Bolts	Class 8.8

**Table 1.** Material of the elements.

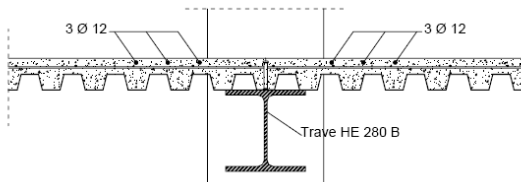
In the following Figures the main elements of the building are shown.



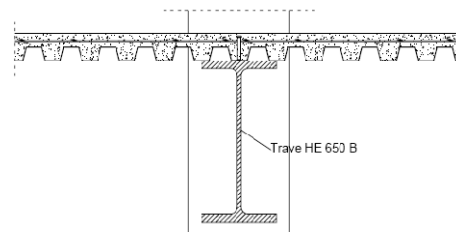
**Figure 5.** Secondary beam HE 200 B.



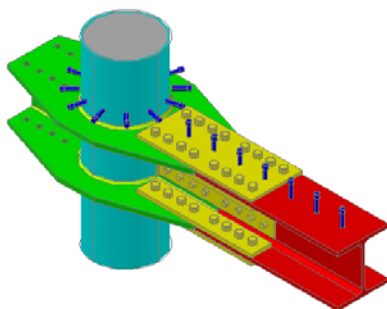
**Figure 6.** Steel sheeting



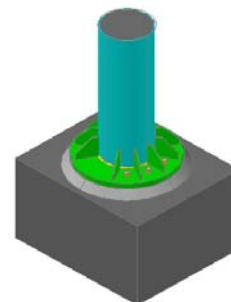
**Figure 7.** Main composite beams (HEB 280)



**Figure 8.** Main composite beam (HEB 650) on open space.



**Figure 9.** Beam-to-column joint.



**Figure 10.** Base joint.

The building is designed with both CHS and CFT columns in S355 and S590 steel grade.

### III Static design of the building

#### III.1 Load analysis

In agreement with EN 1991-1-1[3], the building (offices area) is classified in B category.

##### III.1.1 Dead and live loads

Storey	$G_1$ [kN/m <sup>2</sup> ]	$G_2$ [kN/m <sup>2</sup> ]	Q [kN/m <sup>2</sup> ]
Typical story	2,15	2,32	3,00
Roof	2,15	2,23	2,00*

**Table 2.** Dead loads of the building.

\*Load for accessible roof as the building's location and therefore the snow load is undefined.

##### III.1.2 Wind load

Wind load is calculated in accordance with EN 1991-1-4 [3] and the values are reported in the following table.

$v_{ref,0}$ [m/s]	$A_0$ [m]	$a_s$ [m]	$k_a$ [1/s]	$v_{ref}$ [m/s]	$q_{ref}$ [Pa]	$z_{min}$ [m]	$K_r$	$c_t$	$z_0$ [m]	$c_s * c_d$
27	500	450	0,02	27	455,63	5	0,22	1	0,3	1

Story	Z [m]	$C_s(z)$	$v_m(z)$	$I_v(z)$	$q_p(z)$	$C_{pe+}$	$C_{pe-}$	$q_{tot}(z)$ [N/m <sup>2</sup> ]
1	0,00	0,61	16,36	0,36	583,59	0,75	-0,4	671,130
2	3,50	0,61	16,36	0,36	583,59	0,75	-0,4	671,130
3	7,00	0,68	18,32	0,32	675,79	0,75	-0,4	777,158
4	10,50	0,77	20,68	0,28	793,25	0,75	-0,4	912,239
5	14,00	0,83	22,35	0,26	880,81	0,75	-0,4	1012,927
6	17,50	0,88	23,65	0,25	951,13	0,75	-0,4	1093,798

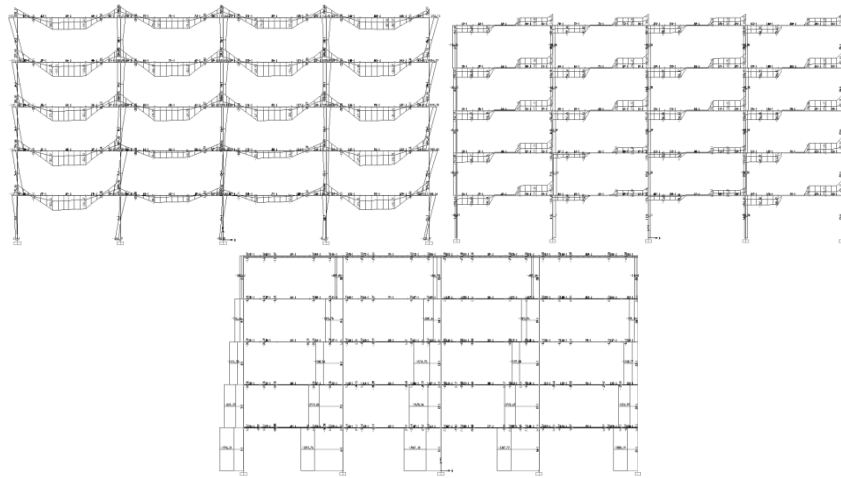
**Table 3.** Wind load.

#### III.2 Analysis and design of the horizontal structural elements

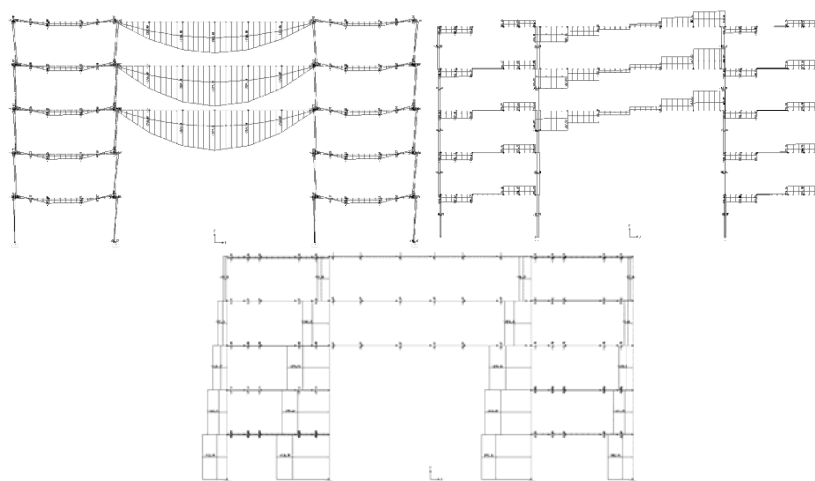
In accordance with EC4-1-1, for static loads, the building's design considers moment resisting frames "A-A"; "B-B" along the y-direction, and pendulum frame "C-C" with concrete shear walls along the x-direction, respectively.

The analysis was carried out by the use of FE program SAP2000 with the stiffness of the composite beams depending on effective width and sign of bending moment as specified in EC8-1 [7] (7.7.2).

The envelope actions in the elements in the frames B-B and C-C are showed below.



**Figure 11.** Moment, shear and axial force diagrams envelope in the moment resisting frame B-B.



**Figure 12.** Moment, shear and axial force diagrams envelope of in the pendulum resisting frame C-C.

Notation: the building is designed with 4 different types of columns, as the differences in the action elements are negligible, only one stiffness is considered in the analysis.

### III.2.1 Verifications of the steel sheeting in the construction phase

The steel sheeting is considered as a continuous beam on 4 simply supports (secondary beams) with spans of 2,67 m. In agreement with EC3-1-3 [5] the steel sheeting is classified as a section of class 4 and the design is in accordance with EC3-1-5 [5].

<i>ULS</i>			
Bending moment	$M_{Ed} = 3,70 \text{ kNm}$	<	$M_{eff,Rd} = 5,68 \text{ kNm}$
Shear	$V_{Ed} = 7,76 \text{ kN}$	<	$V_{Rd} = 55,04 \text{ kN}$
<i>SLS</i>			
Displacement	$\delta_{vert} = 5 \text{ mm}$	<	$\delta_{lim} = 11 \text{ mm}$

**Table 4.** Design of steel sheeting in the construction phase.

### III.2.2 Verifications of the steel sheeting in the work phase

The design of the composite elements is in accordance with EC4-1-1 [6].

<i>ULS</i>			
Bending moment	$M_{Ed} = 9,40 \text{ kNm}$	<	$M_{eff,Rd} = 27,70 \text{ kNm}$
Shear	$V_{Ed} = 14,06 \text{ kN}$	<	$V_{Rd} = 22,37 \text{ kN}$
<i>SLS</i>			
Displacement	$\delta_{vert} = 8 \text{ mm}$	<	$\delta_{lim} = 11 \text{ mm}$

**Table 5.** Design of steel sheeting in work phase.

### III.2.3 Verifications of secondary beams in the construction phase

The beam is considered as a beam on 2 simply supports with span of 8m.

<i>ULS</i>			
Bending moment	$M_{Ed} = 108,9 \text{ kNm}$	<	$M_{pl,Rd} = 176,7 \text{ kNm}$
Lateral-torsional buckling	$M_{Ed,inst} = 108,9 \text{ kNm}$	<	$M_{b,Rd} = 136,7 \text{ kNm}$
Shear	$V_{Ed} = 44,3 \text{ kN}$	<	$V_{pl,Rd} = 394,2 \text{ kN}$
Shear in end sections	$V_{Ed} = 44,3 \text{ kN}$	<	$V_{pl,Rd,mod} = 180 \text{ kN}$
<i>SLS</i>			
Displacement ( $\delta_{lim} = L/250$ )	$\delta_{in} = 8 \text{ mm}$	<	$\delta_{lim} = 32 \text{ mm}$

**Table 6.** Design of secondary beams in the construction phase.

### III.2.4 Verifications of secondary beams in the work phase

<i>ULS</i>		
Bending moment	$M_{Ed} = 256,8 \text{ kNm}$	$< M_{pl,Rd} = 368,6 \text{ kNm}$
Lateral-torsional buckling	Not required in agreement with EC4-1-1 6.4.3 [17]	
Shear	$V_{Ed} = 128,4 \text{ kN}$	$< V_{pl,Rd} = 394,2 \text{ kN}$
Shear in end sections	$V_{Ed} = 128,4 \text{ kN}$	$< V_{pl,Rd,mod} = 180 \text{ kN}$
<i>SLS</i>		
Displacement ( $\delta_{lim} = L/250$ )	$\delta_{ult} = 29 \text{ mm}$	$< \delta_{lim} = 32 \text{ mm}$

**Table 7.** Design of secondary beams in the work phase.

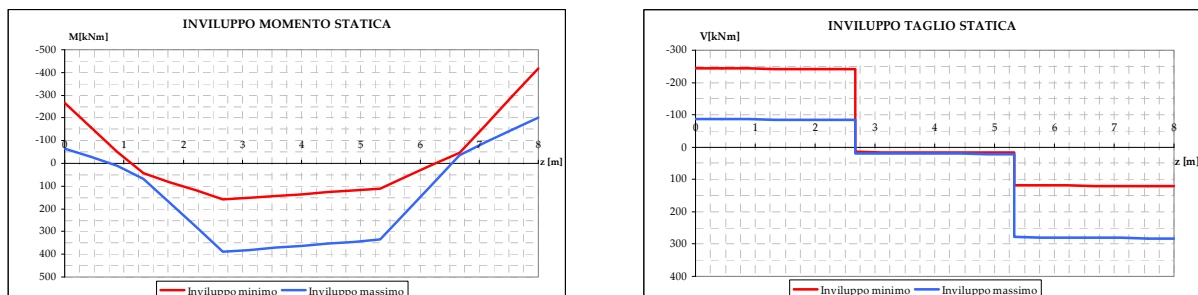
### III.2.5 Verifications of the main beams (L =8 m) in the construction phase

Beams with different effective width of the concrete flange are considered in the design of the building. The exterior beam in the first storey of the C-C frame is the beam with higher actions.

<i>ULS</i>		
Bending moment	$M_{Ed+} = 82,07 \text{ kNm}$	$< M_{pl,Rd} = 421,90 \text{ kNm}$
Bending moment in end sections	$M_{Ed-} =  -163,80  \text{ kNm}$	$< M_{pl,Rd} = 421,90 \text{ kNm}$
Lateral-torsional buckling	$M_{Ed,inst} = 82,07 \text{ kNm}$	$< M_{b,Rd} = 416,30 \text{ kNm}$
Shear	$V_{sd} = 93,26 \text{ kN}$	$< V_{pl,Rd} = 653,03 \text{ kN}$
<i>SLS</i>		
Displacement ( $\delta_{lim} = L/250$ )	$\delta_{in} = 15 \text{ mm}$	$< \delta_{lim} = 32 \text{ mm}$

**Table 8.** Design of exterior main beams of the C-C frame in the construction phase.

### III.2.6 Verifications of main beams (L = 8 m) in the work phase



**Figure 13.** Moment and Shear envelope diagrams.

<i>ULS</i>			
Bending moment	$M_{Ed+} = 390,55 \text{ kNm}$	<	$M_{pl,Rd} = 596,6 \text{ kNm}$
Bending moment in end sections	$M_{Ed-} =  -259,50  \text{ kNm}$	<	$M_{pl,Rd} = 442,0 \text{ kNm}$
Bending moment in end sections	$M_{Ed-} =  -425,87  \text{ kNm}$	<	$M_{pl,Rd} = 478,1 \text{ kNm}$
Lateral-torsional buckling	Not required in agreement with EC4-1-1 6.4.3 [17]		
Shear	$V_{sd} = 283,84 \text{ kN}$	<	$V_{pl,Rd} = 653,03 \text{ kN}$
<i>SLS</i>			
Displacement ( $\delta_{lim} = L/250$ )	$\delta_{in} = 29 \text{ mm}$	<	$\delta_{lim} = 32 \text{ mm}$

Table 9. Design of exterior main beams of the C-C frame in the work phase.

### III.2.7 Verifications of main beams (L = 16 m) over open space in the construction phase

The beam is considered on 2 simply supports with span of 16 m. The resistant section, during the construction phase, is a steel beam, whereas it is a composite beam in the final configuration.

<i>ULS</i>			
Bending moment	$M_{Ed+} = 1163 \text{ kNm}$	<	$M_{pl,Rd} = 2598 \text{ kNm}$
Lateral-torsional buckling	$M_{Ed,inst} = 1163 \text{ kNm}$		$M_{b,Rd} = 1333 \text{ kNm}$
Shear	$V_{Ed} = 246,40 \text{ kN}$	<	$V_{pl,Rd} = 2500 \text{ kN}$
<i>SLS</i>			
Displacement ( $\delta_{lim} = L/250$ )	$\delta_{in} = 15 \text{ mm}$	<	$\delta_{lim} = 64 \text{ mm}$

Table 10. Design of central main beams of the C-C frame in the construction phase.

### III.2.8 Verifications of main beams (L = 16 m) over open space in the work phase

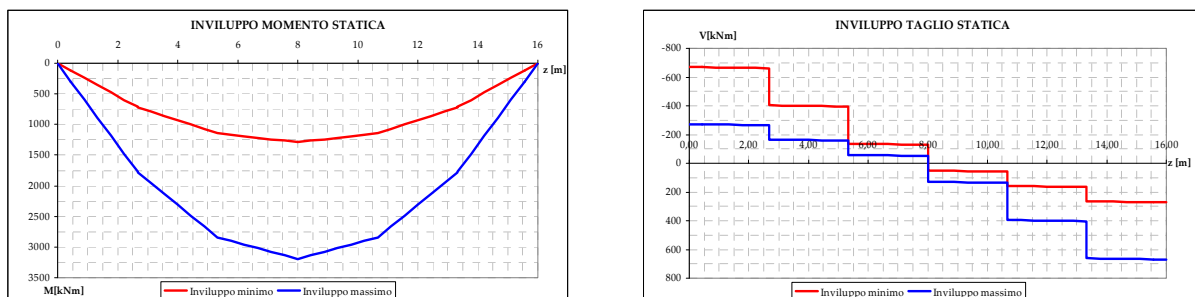


Figure 14. Moment and Shear envelope diagrams.

<b>ULS</b>		
Bending moment	$M_{Ed+} = 3195 \text{ kNm}$	$< M_{pl,Rd} = 3515 \text{ kNm}$
Lateral-torsional buckling	Not required in agreement with EC4-1-1 6.4.3 [17]	
Shear	$V_{Ed} = 670,32 \text{ kN}$	$< V_{pl,Rd} = 2500 \text{ kN}$
<b>SLS</b>		
Displacement ( $\delta_{lim} = L/250$ )	$\delta_{in} = 61 \text{ mm}$	$< \delta_{lim} = 64 \text{ mm}$

**Table 11.** Design of central main beams of the C-C frame in the work phase.

### **III.3 Design and verifications of the structural columns**

Only columns B2 and C2, located in the corner and along the side of the open space, are checked. These columns are the ones with higher actions.

<b>CIRCULAR HOLLOW SECTIONS (CHS)</b>		
Column	<b>S 355</b>	<b>S 590</b>
<b>C-C frame</b>	D=406.4 mm	D=406.4 mm
<b>Open space C2 e C4</b>	t=16 mm	t=16 mm
<b>B2 column</b>	D=406.4 mm t=12 mm	D=406.4 mm t=12 mm
<b>CONCRETE FILLED TUBES (CFT)</b>		
	<b>S 355, C30/37 e 8 <math>\phi</math> 18</b>	<b>S 590, C30/37 e 8 <math>\phi</math> 18</b>
<b>C-C frame</b>	D=355.6 mm	D=355.6 mm
<b>Open space C2 e C4</b>	t=12 mm	t=12 mm
<b>B2 column</b>	D=355.6 mm t=10 mm	D=355.6 mm t=10 mm

**Table 12.** Geometry of the columns.

#### **III.3.1 Design and verifications of CHS columns**

B2 COLUMN S355 (class 2 cross section)								
	Action				Stress		Buckling due to bending and axial compression	
	$N_{Ed}$	$N_{pl,Rd}$	$M_{Ed}$	$M_{pl,Rd}$	$\sigma_{x,max,Sd}$	$\sigma_{xlim}$	Along x	Along y
	[kN]	[kN]	[kNm]	[kNm]	[MPa]	[MPa]		
Comb. 1	3401,12	< 5289,5	941,50	< 315,49			0,780<1	0,741<1
Comb. 2	3736,69	< 241,00	5289,5	< 541,20			0,628<1	0,547<1
	$V_{Ed}$	$V_{pl,Rd}$			$\tau_{max,Sd}$	$\tau_{lim}$		
	[kN]	[kN]			[MPa]	[MPa]		
Shear	102,00	< 1944,00						

B2 COLUMN S590 (class 3 cross section)								
	Action				Stress		Buckling due to bending and axial compression	
	$N_{Ed}$	$N_{pl,Rd}$	$M_{Ed}$	$M_{pl,Rd}$	$\sigma_{x,max,Sd}$	$\sigma_{xlim}$	Along x	Along y
	[kN]	[kN]	[kNm]	[kNm]	[MPa]	[MPa]		
Comb. 1	3401,12		941,50		294,63	< 590	0,513<1	0,513<1
Comb. 2	3736,69		241,00		266,10	< 590	0,447<1	0,447<1
	$V_{Ed}$	$V_{pl,Rd}$			$\tau_{max,Sd}$	$\tau_{lim}$		
	[kN]	[kN]			[MPa]	[MPa]		
Shear	102,00				13,76	< 340,64		

C2 COLUMN S355 (class 1 cross section)								
	Action				Stress		Buckling due to bending and axial compression	
	$N_{Ed}$	$N_{pl,Rd}$	$M_{Ed}$	$M_{pl,Rd}$	$\sigma_{x,max,Sd}$	$\sigma_{xlim}$	Along x	Along y
	[kN]	[kN]	[kNm]	[kNm]	[MPa]	[MPa]		
Comb. 1	4146,69	< 6598,00	187,00	< 466,64			0,801<1	0,740<1
Comb. 2	3736,69	< 6598,00	241,00	< 534,69			0,871<1	0,782<1
	$V_{Ed}$	$V_{pl,Rd}$			$\tau_{max,Sd}$	$\tau_{lim}$		
	[kN]	[kN]			[MPa]	[MPa]		
Shear	136,00	< 2557,43						

C2 COLUMN S590 (class 2 cross section)								
	Action				Stress		Buckling due to bending and axial compression	
	$N_{Ed}$	$N_{pl,Rd}$	$M_{Ed}$	$M_{pl,Rd}$	$\sigma_{x,max,Sd}$	$\sigma_{xlim}$	Along x	Along y
	[kN]	[kN]	[kNm]	[kNm]	[MPa]	[MPa]		
Comb. 1	4146,69	< 11564,00	187,00	< 1231,17			0,498<1	0,457<1
Comb. 2	3736,69	< 11564,00	241,00	< 1299,23			0,566<1	0,511<1
	$V_{Ed}$	$V_{pl,Rd}$			$\tau_{max,Sd}$	$\tau_{lim}$		
	[kN]	[kN]			[MPa]	[MPa]		
Shear	136,00	< 4250,00						

**Table 13.** Design of the columns.

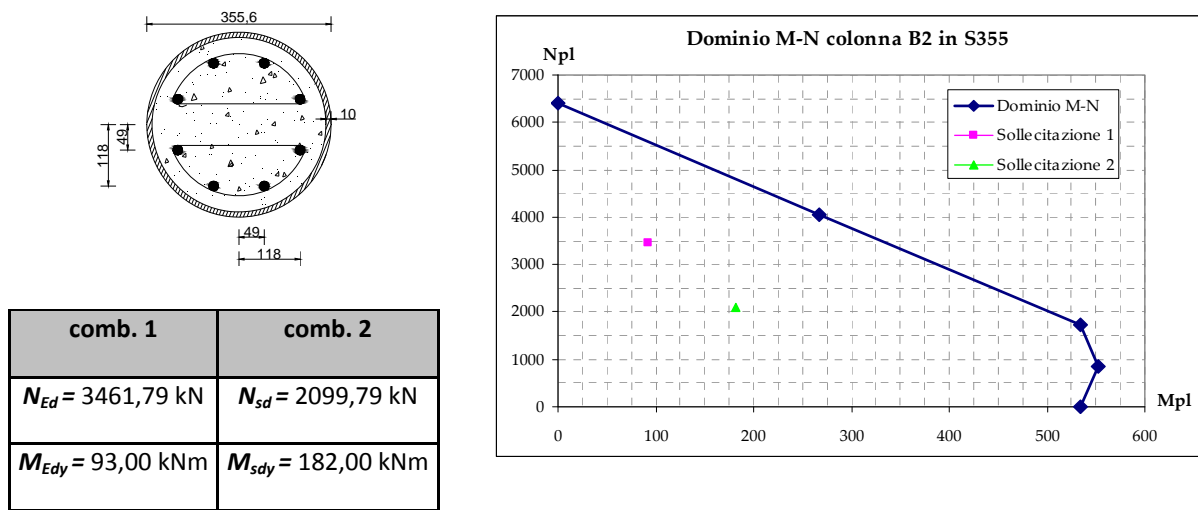
<b>Comb. 1</b>	$N_{Ed, max}$	$M_{Ed}$
<b>Comb. 2</b>	$N_{Ed}$	$M_{Ed, max}$

**Table 14.** Legend of the Table 12

### III.3.2 Design and verifications of CFT columns

These columns are designed using the EC4-1-1 [6] and the dimensions are the same of CHS columns.

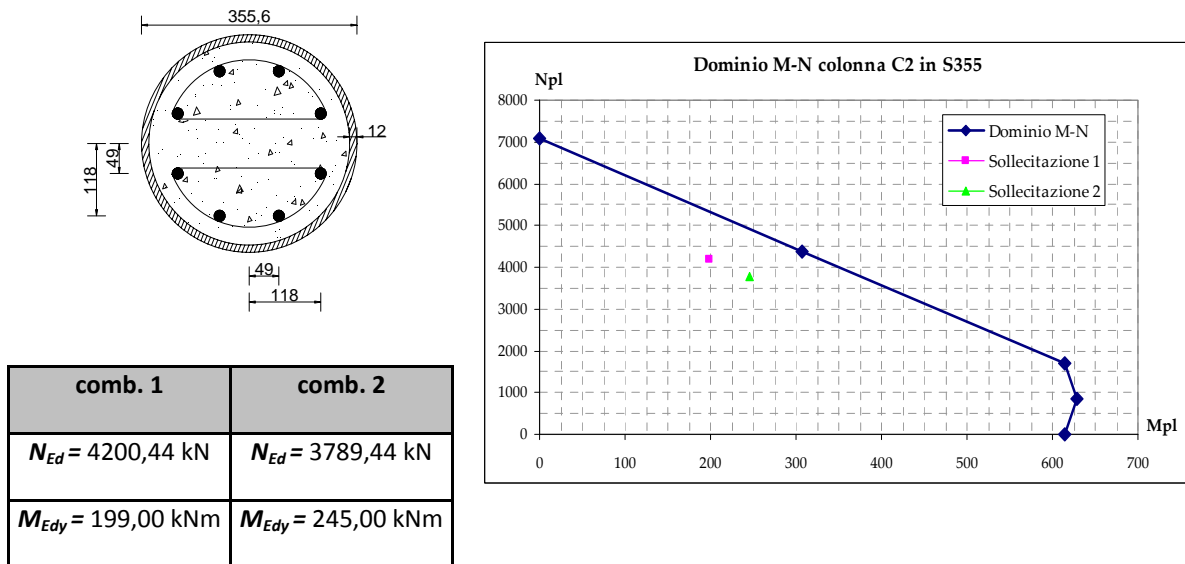
### III.3.2.1 B2 CFT column, S355 (class 2 cross section)



ULS	
Shear	$V_{Ed} = 102,00 \text{ kN} < V_{pl,Rd} = 1422,24 \text{ kN}$

**Table 15. Verification under shear force**

### III.3.2.2 C2 CFT column, S355 (class 1 cross section)



ULS	
Shear	$V_{Ed} = 138,00 \text{ kN} < V_{pl,Rd} = 1696,26 \text{ kN}$

**Table 16. Verification under shear force**

III.3.2.3B2 CFT column, S590 (class 3 cross section)

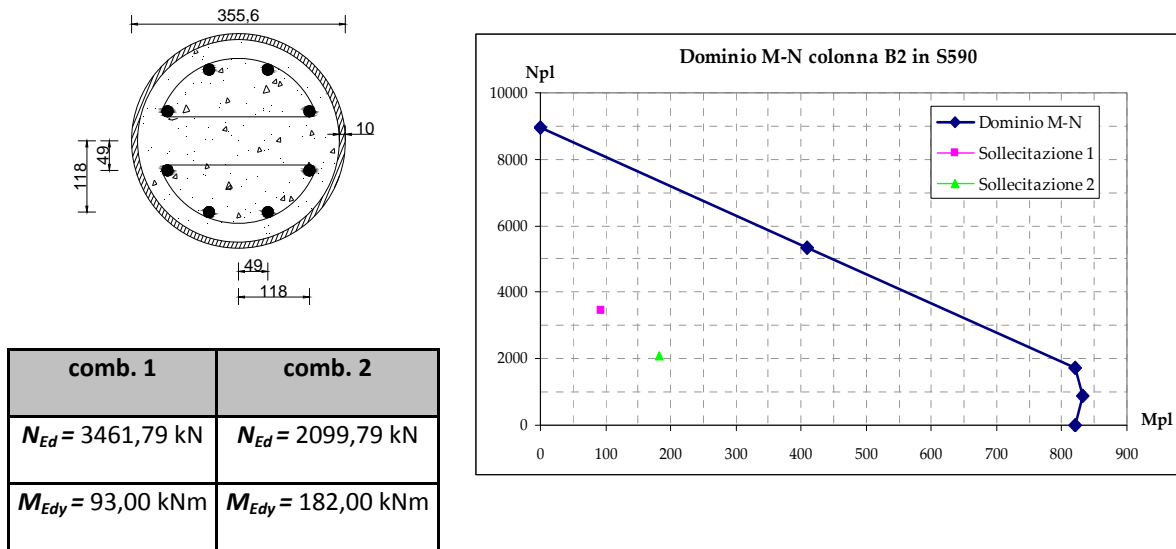


Figure 17. Verification under bending and axial force

ULS		
Shear ( $V_{Ed,max} = 102,00 \text{ kN}$ )	$\tau_{max} = 18,72 \text{ MPa}$	$< \tau_{lim} = 340,64 \text{ MPa}$

Table 17. Verification under shear force

III.3.2.4C2 CFT column, S590 (class 2 cross section)

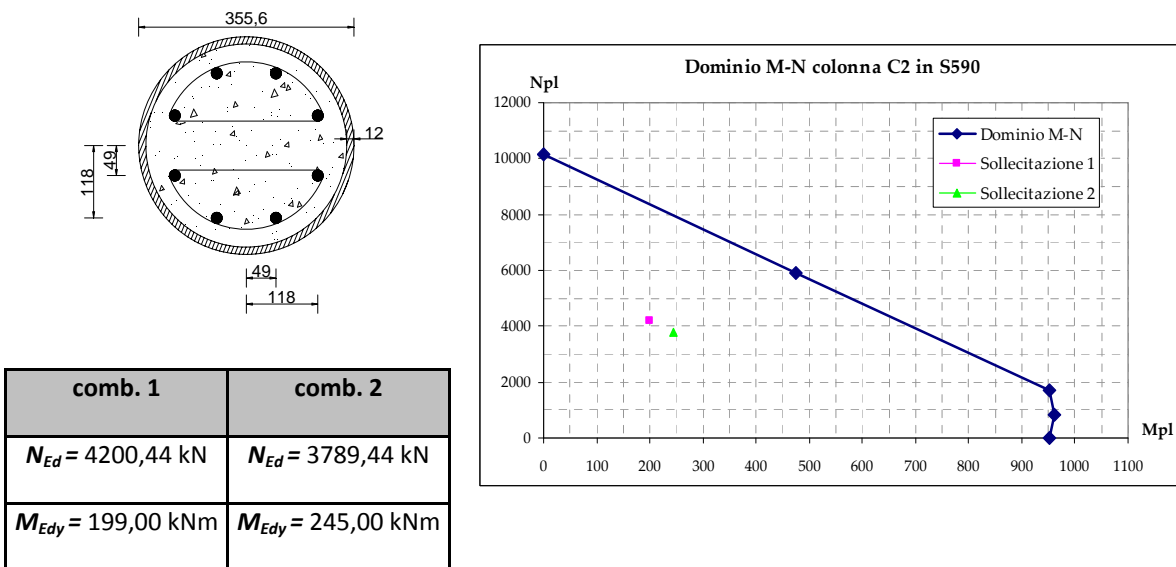


Figure 18. Verification under bending and axial force

<b>ULS</b>		
Shear ( $V_{Ed,max} = 138,00$ kN)	$\tau_{max} = 21,23$ MPa	$< \tau_{lim} = 340,64$ MPa

**Table 18.** Verification under shear force

In the following table the buckling verifications for the columns under bending and axial force. The verifications are in accordance with these formulas:

For S355 steel:  $Eq.1: \frac{N_{Ed}}{\chi N_{pl,Rd}} \leq 1$        $Eq.2: \frac{M_{Ed}}{M_{pl,N,Rd}} = \frac{M_{Ed}}{\mu_{x,d} M_{pl,Rd}} \leq 0,9$

For S590 steel:  $Eq.1: \frac{N_{Ed}}{\chi N_{pl,Rd}} \leq 1$        $Eq.2: \frac{M_{Ed}}{M_{pl,N,Rd}} = \frac{M_{Ed}}{\mu_{x,d} M_{pl,Rd}} \leq 0,8$

B2 and C2 CFT columns under bending and axial compression				
COMBINATIONS	COEFFICIENTS		BUCKLING	
	$\chi$	$\mu_{x,d}$	Eq.1	Eq.2
B2 COLUMN S355 (class 2 cross section)				
COMB.1	0,90	0,56	0,603<1	0,310<0,9
COMB.2	0,87	0,89	0,379<1	0,383<0,9
B2 COLUMN S590 (class 3 cross section)				
COMB.1	0,90	0,73	0,428<1	0,155<0,8
COMB.2	0,87	0,94	0,270<1	0,235<0,8
C2 COLUMN S355 (class 1 cross section)				
COMB.1	0,88	0,44	0,675<1	0,731<0,9
COMB.2	0,79	0,46	0,731<1	0,871<0,9
C2 COLUMN S590 (class 3 cross section)				
COMB.1	0,88	0,66	0,472<1	0,317<1
COMB.2	0,76	0,66	0,490<1	0,390<0,8

**Table 19.** Verification under bending and axial force

## IV Seismic design of the building

The seismic design of the building is performed in accordance with EC8-1 [7]. The main points of this analysis are the following:

- 3D modelling;

- evaluation of limit acceleration of the ground ( $a_{g,limit}$ ) in order to obtain the maximum interstorey drift equal to 1% of interstorey height;
- analysis of the building at the ULS under the limit acceleration of the ground ( $a_{g,limit}$ ).

### IV.1 3D building model

In agreement with EC8-1 [7] 7.7.2 the properties of the material used in the seismic analysis are:

HE650B over the open space : EC8-1, 7.6.3			
$b_{e1}$ [mm]	$b_{e2}$ [mm]	$b_{eff}$ [mm]	$I_1$ [mm <sup>4</sup> ]
600	600	1200	3372869348

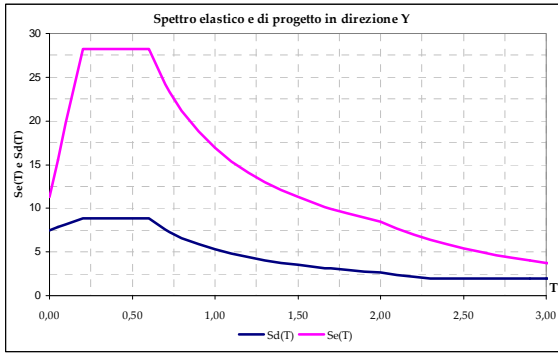
Ieq of the main beams HE 280 B: EC8 - 7.7.2					
Interior beam			Exterior beams		
$I_1$ [mm <sup>4</sup> ]	$I_2$ [mm <sup>4</sup> ]	$I_{eq}$ [mm <sup>4</sup> ]	$I_1$ [mm <sup>4</sup> ]	$I_2$ [mm <sup>4</sup> ]	$I_{eq}$ [mm <sup>4</sup> ]
378085733	224644455	316709222	339667345	209074270	287430115

Column B2 CHS:TUB 406,4 x 12 mm		Column C2 CHS:TUB 406,4 x 16 mm	
$I_a$ [mm <sup>4</sup> ]	$EI_a$ [Nmm <sup>2</sup> ]	$I_a$ [mm <sup>4</sup> ]	$EI_a$ [Nmm <sup>2</sup> ]
289370000	6,077E+13	374490000	7,864E+13

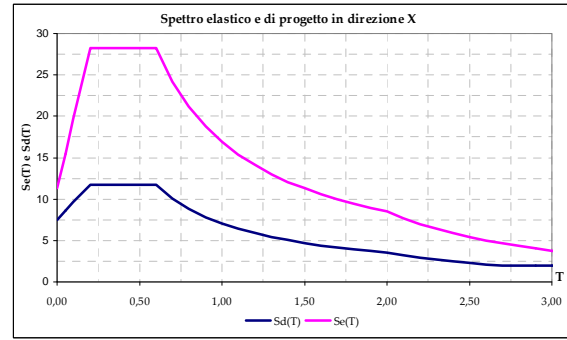
Column B2,CFT:TUB 355,6 x 10 mm						
$E_a$ [N/mm <sup>2</sup> ]	$I_a$ [mm <sup>4</sup> ]	$E_s$ [N/mm <sup>2</sup> ]	$I_s$ [mm <sup>4</sup> ]	$E_{cm}$ [N/mm <sup>2</sup> ]	$I_c$ [mm <sup>4</sup> ]	$(EI)_c$ [Nmm <sup>2</sup> ]
210000	162230000	200000	16666030	33000	606003633	4,266E+13

Column B2,CFT:TUB 355,6 x 10 mm						
$E_a$ [N/mm <sup>2</sup> ]	$I_a$ [mm <sup>4</sup> ]	$E_s$ [N/mm <sup>2</sup> ]	$I_s$ [mm <sup>4</sup> ]	$E_{cm}$ [N/mm <sup>2</sup> ]	$I_c$ [mm <sup>4</sup> ]	$(EI)_c$ [Nmm <sup>2</sup> ]
210000	191390000	200000	16666030	33000	576843895	4,774E+13

**Table 20.** Features of the elements for the analysis by FE program SAP2000



Spectrum along Y (moment resistant frames)

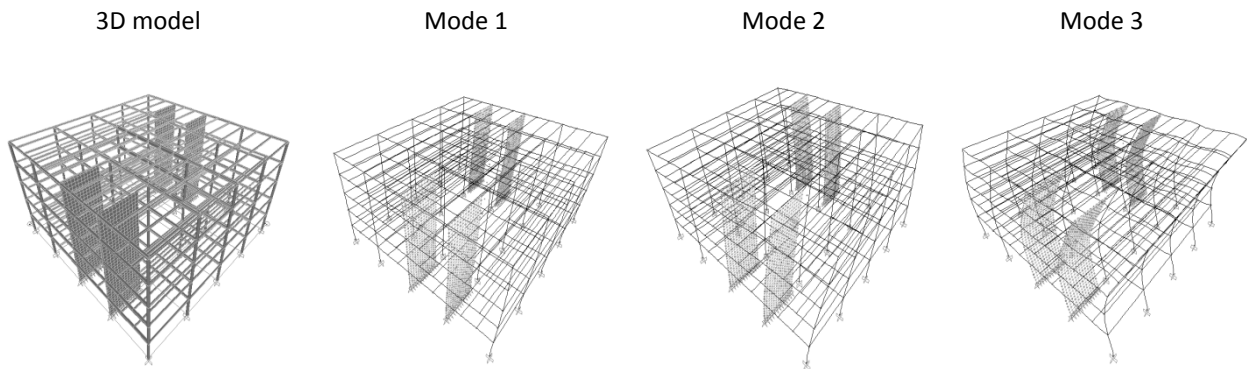


Spectrum along X direction (frames with shear walls)

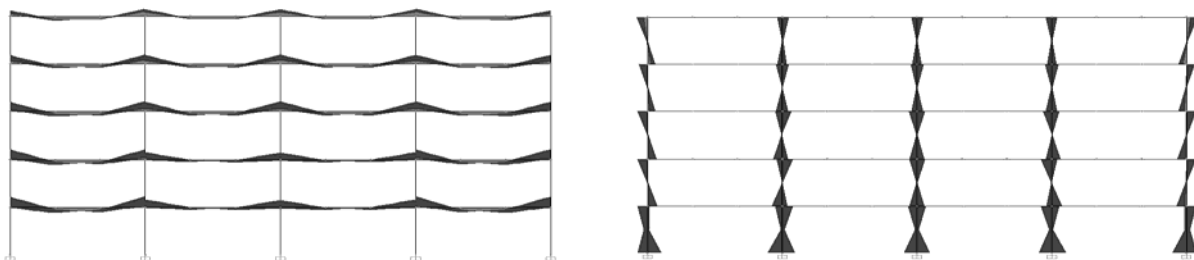
**Figure 19.** Horizontal elastic response spectrum and design spectrum

Storey, <i>i</i>	$W_i$ [kN]	$M_i$ [ton]	$\rho^2$ [m <sup>2</sup> ]	$I_p$ [ton*m <sup>2</sup> ]
1	4125,36	420,53	128,00	53827
2	4125,36	420,53	128,00	53827
3	5415,36	552,02	170,67	94212
4	5415,36	552,02	170,67	94212
5	5628,64	573,77	170,67	97923

**Table 21.** Features for the seismic analysis



**Figure 20.** 3D model and response of the building under seismic analysis



**Figure 21.** Bending moment in the building with CFT columns.

## IV.2 Evaluation of limit acceleration for SLS

The evaluation of the limit accelerations in order to obtain an interstorey drifts equal to 1% of interstorey height is reported in the following Table. These accelerations are used as input for the seismic analysis.

Building with CHS columns	Building with CFT columns
$a_{g,limit} = 0.29g = 2.845 \text{ m/s}^2$	$a_{g,limit} = 0.27g = 2.649 \text{ m/s}^2$

Table 22. Limit acceleration in order to obtain an interstorey drift equal to 1%

## IV.3 Actions and verifications of structural elements

### IV.3.1 Verifications of beams of 3rd storey in the B-B moment-resisting frame

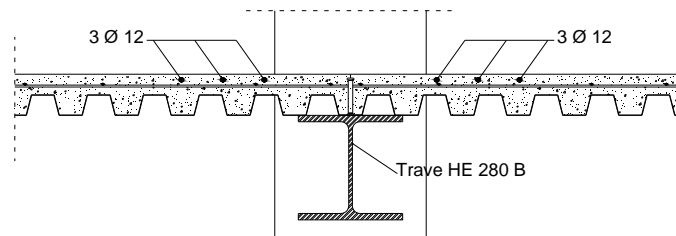


Figure 22. Typical section.

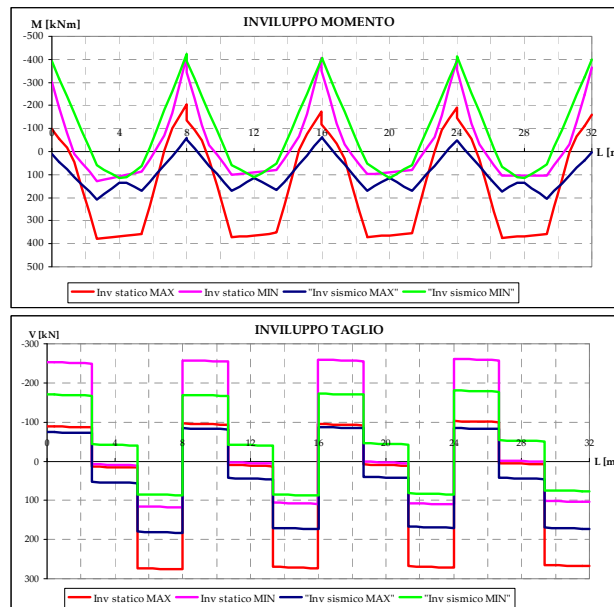


Figure 23. Bending and shear force.

Static loads						
$M_{\max+}$ [kNm]	$M_{\max-}$ [kNm]	$M_{\max}$ [kNm]		$M_{Rd,min}$ [kNm]	$M_{Rd+}$ [kNm]	$M_{Rd,b-}$ [kNm]
379,91	-411,51	<b>411,51</b>	<	<b>478,15</b>	596,64	478,15

Seismic loads						
$M_{\max+}$ [kNm]	$M_{\max-}$ [kNm]	$M_{\max}$ [kNm]		$M_{Rd,min}$ [kNm]	$M_{Rd+}$ [kNm]	$M_{Rd,b-}$ [kNm]
206,68	-422,01	<b>422,01</b>	<	<b>478,15</b>	593,75	478,15

**Table 23.** Bending moment verification

$M_{pl,Rd,dx}$ [kNm]	$M_{pl,Rd,sx}$ [kNm]	$M_{U,Rd,dx}$ [kNm]	$M_{U,Rd,sx}$ [kNm]
478,15	593,75	597,68	742,19

$V_{G,Ed}$ [kN]	+	$V_{M,Ed}$ [kN]	=	$V_{Ed}$ [kN]	<	$0,5 \cdot V_{pl,Rd}$ [kN]	$V_{pl,Rd}$ [kN]
123,85	+	167,48	=	<b>291,34</b>	<	<b>326,51</b>	653,03

**Table 24.** Shear verification

### IV.3.2 Verifications of CHS columns

B2 COLUMN S355 (class 2 cross section)								
	Action				Stress		Buckling due to bending and axial compression	
	$N_{Ed}$	$N_{pl,Rd}$	$M_{Ed}$	$M_{pl,Rd}$	$\sigma_{x,max,Sd}$	$\sigma_{xlim}$		
	[kN]	[kN]	[kNm]	[kNm]	[MPa]	[MPa]	Along x	Along y
Comb. 1	1761,17	< 5289,5	604,30	< 589,4	<		1,021<1	0,767<1
Comb. 2	1449,99	< 5289,5	406,41	< 641,46			0,762<1	0,584<1
	$V_{Ed}$	$V_{pl,Rd}$			$\tau_{max,Sd}$	$\tau_{lim}$		
	[kN]	[kN]			[MPa]	[MPa]		
Shear	281,23	< 972,08						

B2 COLUMN S590 (class 3 cross section)								
	Action				Stress		Buckling due to bending and axial compression	
	$N_{Ed}$	$N_{pl,Rd}$	$M_{Ed}$	$M_{pl,Rd}$	$\sigma_{x,max,Sd}$	$\sigma_{xlim}$	Along x	Along y
	[kN]	[kN]	[kNm]	[kNm]	[MPa]	[MPa]		
Comb. 1	1761,17		604,30		550,94	< 590	0,827<1	0,827<1
Comb. 2	1449,99		406,41		386,09	590	0,596<1	0,596<1
	$V_{Ed}$	$V_{pl,Rd}$			$\tau_{max,Sd}$	$\tau_{lim}$		
	[kN]	[kN]			[MPa]	[MPa]		
Shear	281,23	< 1268,87						

C2 COLUMN S355 (class 1 cross section)								
	Action				Stress		Buckling due to bending and axial compression	
	$N_{Ed}$	$N_{pl,Rd}$	$M_{Ed}$	$M_{pl,Rd}$	$\sigma_{x,max,Sd}$	$\sigma_{xlim}$	Along x	Along y
	[kN]	[kN]	[kNm]	[kNm]	[MPa]	[MPa]		
Comb. 1	2474,27	< 6598,00	772,15	< 744,24		<	1,053<1	0,796<1
Comb. 2	2191,18	< 6598,00	332,90	< 791,23			0,704<1	0,584<1
	$V_{Ed}$	$V_{pl,Rd}$			$\tau_{max,Sd}$	$\tau_{lim}$		
	[kN]	[kN]			[MPa]	[MPa]		
Shear	321,04	< 1278,72						

C2 COLUMN S590 (class 2 cross section)								
	Action				Stress		Buckling due to bending and axial compression	
	$N_{Ed}$	$N_{pl,Rd}$	$M_{Ed}$	$M_{pl,Rd}$	$\sigma_{x,max,Sd}$	$\sigma_{xlim}$		
	[kN]	[kN]	[kNm]	[kNm]	[MPa]	[MPa]	Along x	Along y
Comb. 1	2474,27	< 11564,00	772,15	< 1439,6	<		0,667<1	0,502<1
Comb. 2	2191,18	< 11564,00	332,90	< 1439,60			0,458<1	0,384<1
	$V_{Ed}$	$V_{pl,Rd}$			$\tau_{max,Sd}$	$\tau_{lim}$		
	[kN]	[kN]			[MPa]	[MPa]		
Shear	321,04	< 2125,19						

**Table 25.** Design of the columns.

<b>Comb. 1</b>	$N_{Ed, max}$	$M_{Ed}$
<b>Comb. 2</b>	$N_{Ed}$	$M_{Ed, max}$

**Table 26.** Legend of the Table 25

### Capacity design of the joints, EC8-1, 4.4.2.2

Capacity design of the joints in order to obtain the hierarchy of resistance of the components

Joint of the first storey					
Column B2; CHS; S355			Beams of the joint		
$M_{Rd,c+}$	589,48	kNm	$M_{Rd,b+}$	593,75	kNm
$M_{rd,c-}$	589,48	kNm	$M_{rd,b-}$	478,15	kNm
$\Sigma(M_{rd,c})$	<b>1178,95</b>	<b>kNm</b>	$1,3\Sigma(M_{rd,b})$	<b>1393,47</b>	<b>kNm</b>
Not satisfied					
Joint of the second storey					
Column B2; CHS; S355			Beams of the joint		
$M_{Rd,c+}$	641,46	kNm	$M_{Rd,b+}$	593,75	kNm
$M_{rd,c-}$	641,46	kNm	$M_{rd,b-}$	478,15	kNm
$\Sigma(M_{rd,c})$	<b>1282,93</b>	<b>kNm</b>	$1,3\Sigma(M_{rd,b})$	<b>1393,47</b>	<b>kNm</b>
Not satisfied					

Joint of any storey					
Column B2; CHS; S590			Beams of the joint		
$M_{Rd,c+}$	840,16	kNm	$M_{Rd,b+}$	593,75	kNm
$M_{rd,c-}$	840,16	kNm	$M_{rd,b-}$	478,15	kNm
$\Sigma(M_{rd,c})$	<b>1680,32</b>	<b>kNm</b>	$1,3\Sigma(M_{rd,b})$	<b>1393,47</b>	<b>kNm</b>
Satisfied					

Joint of the first storey					
Column C2; CHS; S355			Beams of the joint		
$M_{Rd,c+}$	744,24	kNm	$M_{Rd,b+}$	593,75	kNm
$M_{rd,c-}$	744,24	kNm	$M_{rd,b-}$	0	kNm
$\Sigma(M_{rd,c})$	<b>1488,48</b>	<b>kNm</b>	$1,3\Sigma(M_{rd,b})$	<b>771,88</b>	<b>kNm</b>
Satisfied					

Joint of second storey					
Column C2; CHS; S355			Beams of the joint		
$M_{Rd,c+}$	791,23	kNm	$M_{Rd,b+}$	593,75	kNm
$M_{Rd,c-}$	791,23	kNm	$M_{Rd,b-}$	0	kNm
$\Sigma(M_{Rd,c})$	1582,45	kNm	$1,3\Sigma(M_{Rd,b})$	771,88	kNm
Satisfied					

Joint of any storey					
Column C2; CHS; S590			Beams of the joint		
$M_{Rd,c+}$	1439,60	kNm	$M_{Rd,b+}$	593,75	kNm
$M_{Rd,c-}$	1439,60	kNm	$M_{Rd,b-}$	0	kNm
$\Sigma(M_{Rd,c})$	2879,20	kNm	$1,3\Sigma(M_{Rd,b})$	771,88	kNm
Satisfied					

Table 27. Capacity design of the joints.

### IV.3.3 Verifications of CFT columns

#### IV.3.3.1 B2 CFT column, S355 (class 2 cross section)

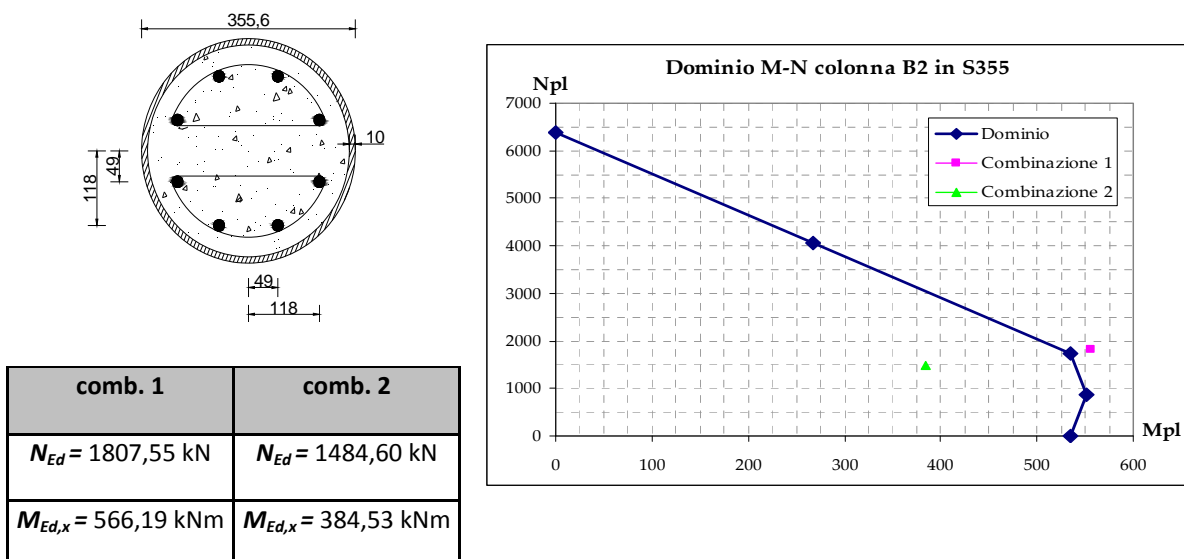


Figure 24. Verification under bending and axial force

ULS		
Shear	$V_{Ed,max} = 237,94 \text{ kN}$	$< 0,5V_{pl,Rd} = 711,12 \text{ kN}$

Table 28. Verification under shear force

IV.3.3.2 C2 CFT column, S355 (class 1 cross section)

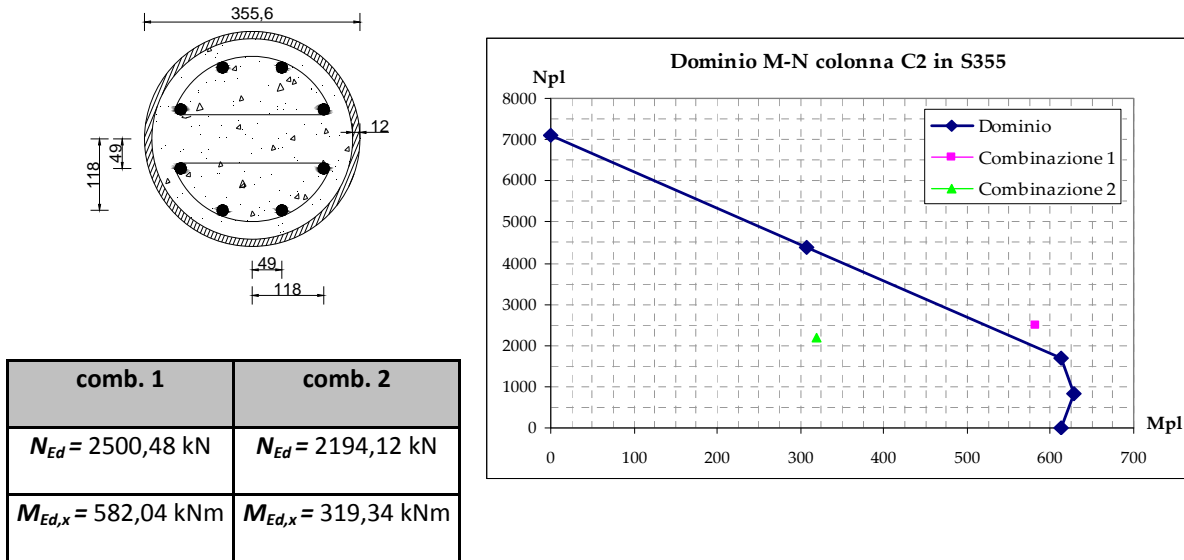


Figure 25. Verification under bending and axial force

ULS		
Shear	$V_{Ed,max} = 269,15 \text{ kN}$	$< 0,5V_{pl,Rd} = 848,13 \text{ kN}$

Table 29. Verification under shear force

IV.3.3.3 B2 CFT column, S590 (class 3 cross section)

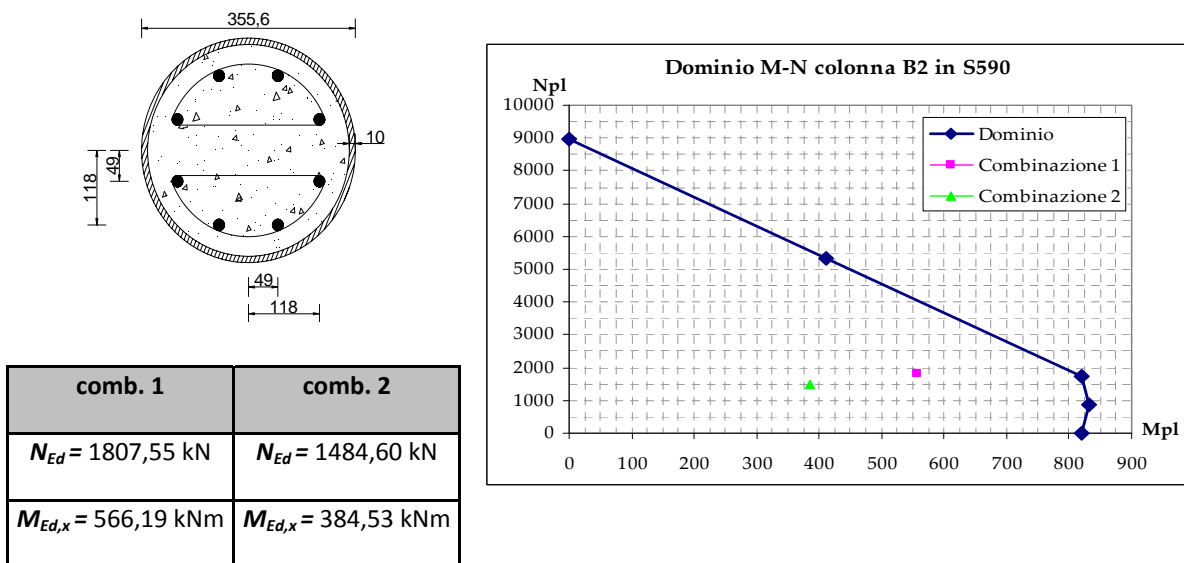
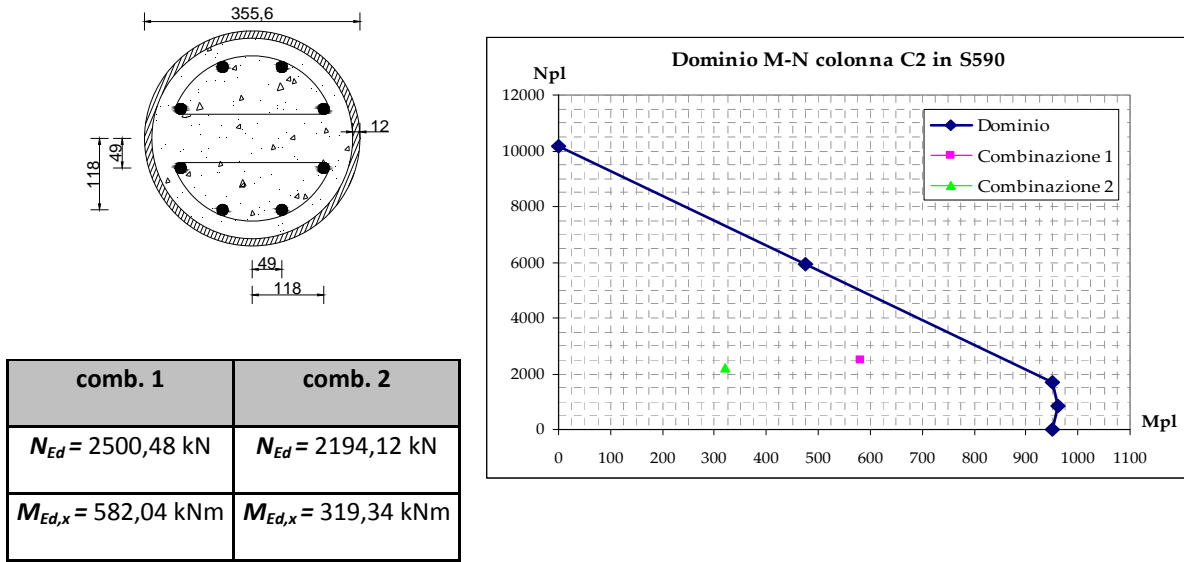


Figure 26. Verification under bending and axial force

ULS		
Shear	$V_{Ed,max} = 273,94 \text{ kN}$	$< 0,5V_{pl,Rd} = 1181,87 \text{ kN}$

**Table 30.** Verification under shear force

IV.3.3.4 C2 CFT column, S590 (class 2 cross section)



**Figure 27.** Verification under bending and axial force

ULS		
Shear	$V_{Ed,max} = 269,15 \text{ kN}$	$< 0,5V_{pl,Rd} = 1409,56 \text{ kN}$

**Table 31.** Verification under shear force

In the following table there are the buckling verifications for the column under bending and axial force. The verifications are in accordance with these formulas:

For S355 steel: Eq.1:  $\frac{N_{Ed}}{\chi N_{pl,Rd}} \leq 1$

Eq.2:  $\frac{M_{Ed}}{M_{pl,N,Rd}} = \frac{M_{Ed}}{\mu_{x,d} M_{pl,Rd}} \leq 0,9$

For S590 steel: Eq.1:  $\frac{N_{Ed}}{\chi N_{pl,Rd}} \leq 1$

Eq.2:  $\frac{M_{Ed}}{M_{pl,N,Rd}} = \frac{M_{Ed}}{\mu_{x,d} M_{pl,Rd}} \leq 0,8$

B2 and C2 CFT columns under bending and axial compression				
COMBINATIONS	COEFFICIENTS		BUCKLING	
	$\chi$	$\mu_{x,d}$	Eq.1	Eq.2
B2 COLUMN S355 (class 2 cross section)				
COMB.1	0,90	0,97	0,316<1	1,074<0,9
COMB.2	0,86	1	0,270<1	0,720<0,9
B2 COLUMN S590 (class 3 cross section)				
COMB.1	0,90	0,99	0,224<1	0,683<0,8
COMB.2	0,86	1	0,193<1	0,469<0,8
C2 COLUMN S355 (class a cross section)				
COMB.1	0,87	0,81	0,405<1	1,165<0,9
COMB.2	0,77	0,84	0,403<1	0,616<0,9
C2 COLUMN S590 (class 3 cross section)				
COMB.1	0,87	0,90	0,284<1	0,682<0,8
COMB.2	0,72	0,92	0,298<1	0,365<0,8

**Table 32.** Verification under bending and axial force

#### IV.3.4 Capacity design of the joints, EC8-1, 4.4.2.2

Capacity design of the joints in order to obtain hierarchy of resistance of the components

Joint of the first storey					
Column B2; CHS; S355			Beams of the joint		
$M_{Rd,c+}$	525,02	kNm	$M_{Rd,b+}$	593,75	kNm
$M_{rd,c-}$	525,02	kNm	$M_{rd,b-}$	478,15	kNm
$\Sigma(M_{rd,c})$	<b>1050,04</b>	<b>kNm</b>	$1,3\Sigma(M_{rd,b})$	<b>1393,47</b>	<b>kNm</b>
<b>Not satisfied</b>					
Joint of the second storey					
Column B2; CHS; S355			Beams of the joint		
$M_{Rd,c+}$	534,07	kNm	$M_{Rd,b+}$	593,75	kNm
$M_{rd,c-}$	534,07	kNm	$M_{rd,b-}$	478,15	kNm
$\Sigma(M_{rd,c})$	<b>1068,15</b>	<b>kNm</b>	$1,3\Sigma(M_{rd,b})$	<b>1393,47</b>	<b>kNm</b>
<b>Not satisfied</b>					

Joint of the first storey					
Column B2; CHS; S590			Beams of the joint		
$M_{Rd,c+}$	811,39	kNm	$M_{Rd,b+}$	593,75	kNm
$M_{rd,c-}$	811,39	kNm	$M_{rd,b-}$	478,15	kNm
$\Sigma(M_{rd,c})$	<b>1622,79</b>	<b>kNm</b>	$1,3\Sigma(M_{rd,b})$	<b>1393,47</b>	<b>kNm</b>
<b>Satisfied</b>					
Joint of the second storey					
Column B2; CHS; S590			Beams of the joint		
$M_{Rd,c+}$	820,37	kNm	$M_{Rd,b+}$	593,75	kNm
$M_{rd,c-}$	820,37	kNm	$M_{rd,b-}$	478,15	kNm
$\Sigma(M_{rd,c})$	<b>1640,75</b>	<b>kNm</b>	$1,3\Sigma(M_{rd,b})$	<b>1393,47</b>	<b>kNm</b>
<b>Satisfied</b>					

Joint of the first storey					
Column C2; CHS; S355			Beams of the joint		
$M_{Rd,c+}$	521,39	kNm	$M_{Rd,b+}$	593,75	kNm
$M_{Rd,c-}$	521,39	kNm	$M_{Rd,b-}$	0	kNm
$\Sigma(M_{Rd,c})$	<b>1042,79</b>	<b>kNm</b>	$1,3\Sigma(M_{Rd,b})$	<b>771,88</b>	<b>kNm</b>
Satisfied					
Joint of the second storey					
Column C2; CHS; S355			Beams of the joint		
$M_{Rd,c+}$	556,14	kNm	$M_{Rd,b+}$	593,75	kNm
$M_{Rd,c-}$	556,14	kNm	$M_{Rd,b-}$	0	kNm
$\Sigma(M_{Rd,c})$	<b>1112,28</b>	<b>kNm</b>	$1,3\Sigma(M_{Rd,b})$	<b>771,88</b>	<b>kNm</b>
Satisfied					

Joint of the first storey					
Column C2; CHS; S590			Beams of the joint		
$M_{Rd,c+}$	860,13	kNm	$M_{Rd,b+}$	593,75	kNm
$M_{Rd,c-}$	860,13	kNm	$M_{Rd,b-}$	0	kNm
$\Sigma(M_{Rd,c})$	<b>1720,26</b>	<b>kNm</b>	$1,3\Sigma(M_{Rd,b})$	<b>771,88</b>	<b>kNm</b>
Satisfied					
Joint of the second storey					
Column C2; CHS; S590			Beams of the joint		
$M_{Rd,c+}$	894,56	kNm	$M_{Rd,b+}$	593,75	kNm
$M_{Rd,c-}$	894,56	kNm	$M_{Rd,b-}$	0	kNm
$\Sigma(M_{Rd,c})$	<b>1789</b>	<b>kNm</b>	$1,3\Sigma(M_{Rd,b})$	<b>771,88</b>	<b>kNm</b>
Satisfied					

**Table 33.** Capacity design of the joints.

In the following table the resume of the results in seismic analysis for CHS columns is reported.

	<b>COLUMNS S355</b>	<b>COLUMNS S590Q</b>
<b>General Columns [B2]</b>	<b>TUB 406,4 x 12 mm</b>	<b>TUB 406,4 x 12 mm</b>
<b>Columns open-space [C2]</b>	<b>TUB 406,4 x 16 mm</b>	<b>TUB 406,4 x 16 mm</b>
<b>Damage Limitation <math>a_{g,limit}</math></b>	0,29 g	0,29g
<b>Ultimate Limit State <math>a_g = 0,2g</math></b>	COL B2: NOT satisfied: <ul style="list-style-type: none"> <li>▪ capacity design requirement</li> </ul>	COL B2 S590: OK
	COL C2: OK	COL C2 S590: OK
<b>Ultimate Limit State <math>a_g = 0,29g</math></b>	COL B2: NOT satisfied: <ul style="list-style-type: none"> <li>▪ resistance verification</li> <li>▪ instability verification</li> <li>▪ capacity design requirement</li> </ul>	COL B2 S590: OK
	COL C2: NOT satisfied: <ul style="list-style-type: none"> <li>▪ resistance verification</li> <li>▪ instability verification</li> </ul>	COL C2 S590: OK

**Table 34.** Resume of CHS column in seismic analysis.

In the following table the resume of the results in seismic analysis for CFT columns is reported

	<b>COLUMNS S355 and CONC 30/37</b>	<b>COLUMNS S590Q and CONC 30/37</b>
<b>General Columns [B2]</b>	<b>TUB 355,6 x 10 mm + 8 <math>\emptyset</math> 18</b>	<b>TUB 355,6 x 10 mm + 8 <math>\emptyset</math> 18</b>
<b>Columns open-space [C2]</b>	<b>TUB 355,6 x 12 mm + 8 <math>\emptyset</math> 18</b>	<b>TUB 355,6 x 12 mm + 8 <math>\emptyset</math> 18</b>
<b>Damage Limitation <math>a_{g,limit}</math></b>	0,27 g	0,27g
<b>Ultimate Limit State <math>a_g = 0,27g</math></b>	COL B2: NOT satisfied: <ul style="list-style-type: none"> <li>▪ resistance verification</li> <li>▪ instability verification</li> <li>▪ capacity design requirement</li> </ul>	COL B2: OK
	COL C2: NOT satisfied: <ul style="list-style-type: none"> <li>▪ resistance verification</li> <li>▪ instability verification</li> </ul>	COL C2: OK

**Table 35.** Resume of CFT column in seismic analysis.

### IV.3.5 Seismic design of the beam-to-column joint

The beam-to column joint is designed in order to be a rigid and full strength joint in agreement with EC3-1-8 [5]. The geometry of the designed joints is reported in the following Figures.

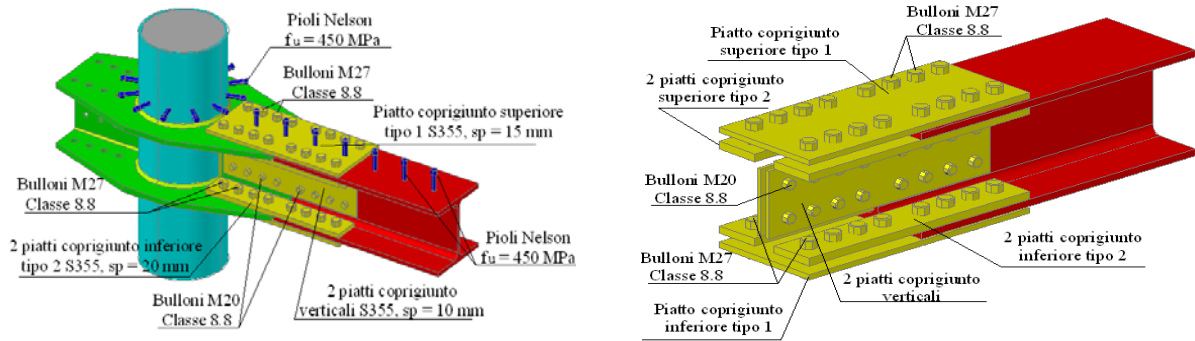


Figure 28. Model of the rigid and full strength joint

#### IV.3.5.1 Components of the joints

- mechanisms in compression concrete slab

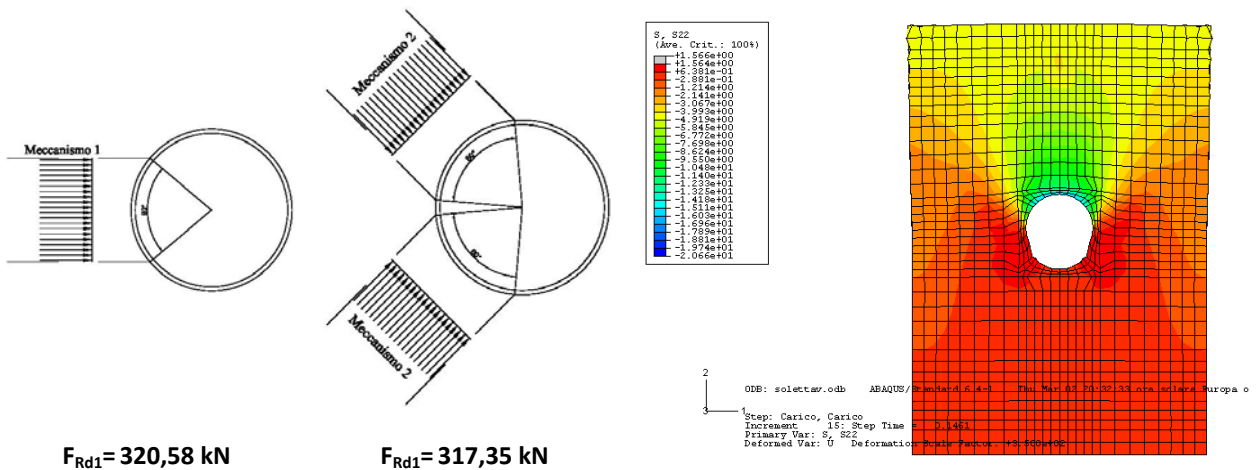


Figure 29. mechanism in compression concrete slab.

$$F_{Rd} = F_{Rd1} + F_{Rd2} = 637,93 \text{ kN.}$$

- upper component of the joint in tension

	Area	Action	Action in the hierarchy of resistance
Flange of the beam	5040 mm <sup>2</sup>	1386,00 kN	1905,75 kN
Portion of the web	501 mm <sup>2</sup>	138,55 kN	189,44 kN
Area in tension	5541 mm <sup>2</sup>	1661,55 kN	2284,63 kN

**Table 36.** Force in the upper plate in compression

Type of resistance	Formula of resistance	Resistance
Shear resistance of the bolts	$F_{v,Rd} = n_b n_{shear, layers} \frac{\alpha_v f_{ub} A_{res}}{\gamma_{M2}} = 2 \cdot 8 \frac{0,6 \cdot 800 \cdot 459}{1,25}$	2820,100kN
Tension resistance of the plate bolted to beam	$N_{u,Rd} = \frac{0,9 \cdot A_{net} \cdot f_u}{\gamma_{M2}} = \frac{0,9 \cdot 5500 \cdot 510}{1,25}$	2386,80kN
Tension resistance of the plate welded to column	$N_{u,Rd} = \frac{0,9 A_{net} f_u}{\gamma_{M2}} = \frac{0,9 \cdot 7429 \cdot 510}{1,25}$	2727,93kN
Bearing resistance of the flange of the beam	$F_{b,Rd} = 2347,20 \text{ kN}$	2347,20 kN
Bearing resistance of the plate bolted to beam	$F_{b,Rd} = 4284,00 \text{ kN}$	4284,00 kN
Bearing resistance of the plate welded to column	$F_{b,Rd} = 2815,20 \text{ kN}$	2815,20 kN
		<b>R<sub>min</sub> = 2347,20 kN</b>

**Table 37.** Resistance of the upper elements of the joint

The upper plate is safe because the minimum resistance is higher than the action in the upper plate:

$$R_{\min} = 2347,20 \text{ kN} > 2284,63 \text{ kN}$$

- vertical component of the of the joint in tension

The tension resistance of the web beam is the following:

$$N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} = \frac{2058 \text{ mm}^2 \cdot 275 \text{ MPa}}{1} = 565,95 \text{ kN} \leq N_{u,Rd} = \frac{0,9 \cdot A_{net} \cdot f_u}{\gamma_{M2}} = \frac{0,9 \cdot 2078 \text{ mm}^2 \cdot 430 \text{ MPa}}{1,25} = 643,30 \text{ kN}$$

The tension resistance of the plate bolted to the web of the beam is the following:

- tension in the web of the beam due to bending moment

$$F_{v,Ed} = \frac{1,1 \cdot \gamma_{ov} \cdot A_w \cdot f_y}{\gamma_{M0}} = \frac{1,1 \cdot 1,25 \cdot 2058 \text{ mm}^2 \cdot 275 \text{ MPa}}{1} = 778,18 \text{ kN}$$

- tension in the web equal to the shear in web of the beam

$$V_{Ed} = 1,1 \cdot V_{Sd} = 320,47 \text{ kN}$$

- maximum tension

$$F_{v,Ed,tot} = \sqrt{(V_{Ed}^2 + F_{v,Ed2})} = 841,59 \text{ kN}$$

Type of resistance	Formula of resistance	Resistance
Shear resistance of the bolts	$F_{v,Rd} = n_b n_{shear, layers} \frac{\alpha_v f_{ub} A_{res}}{\gamma_{M2}} = 2 \cdot 8 \frac{0,6 \cdot 800 \cdot 245}{1,25}$	1505.280 kN
Tension resistance of the plate bolted to web of the beam	$N_{u,Rd} = \frac{0,9 \cdot A_{net} \cdot f_u}{\gamma_{M2}} = \frac{0,9 \cdot 2720 \cdot 510}{1,25}$	998,78 kN
Bearing resistance of the web of the beam	$F_{b,Rd} = 1440,73 \text{ kN}$	1140,73 kN
Bearing resistance of the plate bolted to web of the beam	$F_{b,Rd} = 2472,73 \text{ kN}$	2472,73 kN
Bearing resistance of the plate welded to column	$F_{b,Rd} = 1730,91 \text{ kN}$	1730,91 kN
		<b>R<sub>min</sub> = 998,78 kN</b>

**Table 38.** Tension resistance of the plate bolted to the web of beam

The resistance of vertical elements of the joint is higher than the action, due to hierarchy resistance, in the web of beam:

$$R_{\min} = 998,78 \text{ kN} > 841,49 \text{ kN}$$

- lower component of the joint in tension

For the lower flange of the beam the results are the same of the upper flange of the beam.

The resistances of the components of the joint are written in following table:

concrete slab in compression	637,93	kN
upper component of the joint in tension	2347,20	kN
vertical component of the joint in tension	998,78	kN
lower component of the joint in tension	2347,20	kN

**Table 39.** Resistance of the elements of the joint

In agreement with EC3-1-8 [5] the stiffness of the joint by the use of the stiffness of the components is evaluated.

Stiffness of the steel components	Stiffness of concrete slab in compression
$K = \frac{E \cdot A}{L_{def}}$	$K = \frac{E_{cm}}{L_{def}} \cdot \frac{F_{Rd}}{f_{cd}}$

**Table 40.** Formulas to evaluate the stiffness

In the table are reported the stiffness of components for sagging bending moment on the joint.

<i>beam-column joint, positive bending moment</i>				
Component	$F_{Rd,i}$ [kN]	$K_i$ [kN/mm]	$k_i=K_i/E$ [mm]	$z_i$ [mm]
Concrete slab in compression	637,93	5920	28	41
Upper plate in compression	2347,20	26427	126	116,5
Vertical plate in compression	180,43	1851	9	150
Vertical plate in tension	818,36	8397	40	272
Lower plate in tension	2347,20	26427	126	383,5

**Table 41.** Stiffness of the components of the joint for sagging bending moment

Evaluation of the equivalent arm in agreement with EC3-1-8 6.3.3.1 (3) [5]

Formula	$z_{eq,compr}$ [mm]	$z_{eq,traz}$ [mm]	$z_{nom}$ [mm]
$z_{eq} = \frac{\sum_i (k_i \cdot z_i^2)}{\sum_i (k_i \cdot z_i)}$	114 mm	363 mm	249 mm

**Table 42.** Equivalent lever arms

Check of the hierarchy of resistance of the joint

$F_{Rd,comp}^*$	$F_{Rd,traz}^*$	$M_{Rd+} = Z_{mom+} \cdot \min(F_{Rd,comp}; F_{Rd,traz})$		$M_{Rd,b+}^{**}$
3165,56 kN	3165,56 kN	$M_{Rd+} = 249 \text{ mm} \cdot 3165,56 \text{ kN} = 788,27 \text{ kNm}$	>	593,75 kNm
* sum of the resistance of components				
** resistant plastic moment of the beam				

**Table 43.** Check of the hierarchy of resistance of the joint

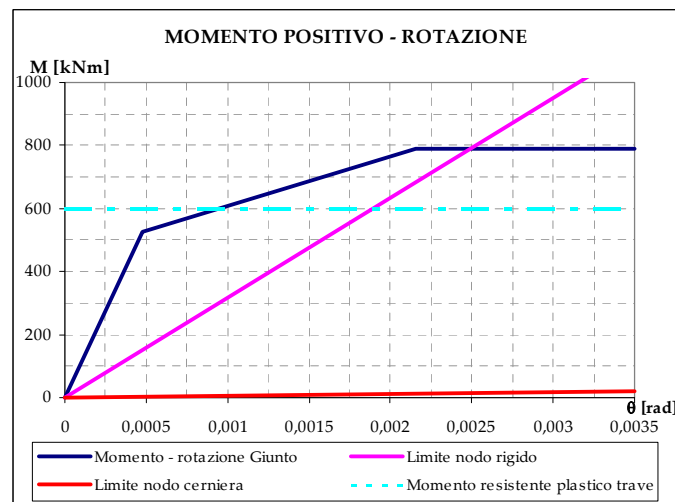
Evaluation of the equivalent stiffness of the springs:

Formula	Stiffness	$K_{eq}$ [kN/mm]	$k_{eq} = K_{eq}/E$ [mm]
$K_{eq} = \frac{\sum_{i=1}^n (k_i \cdot \delta_i^2)}{Z^2}$	Spring in compression	37133	177
	Spring in tension	34340	164

**Table 44.** Stiffness of the spring of the joint model

Initial joint stiffness:	$S_{j,in} = \frac{Z^2}{\sum_{i=1}^2 \left( \frac{1}{K_{eq,i}} \right)} = 1106298 \text{ kNm}$
Secant stiffness:	$S_j = \frac{Z^2}{\mu \cdot \sum_{i=1}^2 \left( \frac{1}{K_{eq,i}} \right)} = \frac{Z^2}{(1,5)^{2,7} \cdot \sum_{i=1}^2 \left( \frac{1}{K_{eq,i}} \right)} = 365013 \text{ kNm}$
Limit stiffness	$S_{j,lim} = \frac{25 \cdot E \cdot I_{l,b}}{L_b} = \frac{25 \cdot 210000 \text{ MPa} \cdot 482270232 \text{ mm}^4}{8000 \text{ mm}} = 316490 \text{ kNm}$

**Table 45.** Stiffness of the joint model



**Figure 30.** Moment rotation diagram of the joint for sagging bending moment.

The only differences between joint under sagging and hogging bending moment are the reinforcement steel in the concrete slabs. In agreement with EC4-1-1 A.2.1.1 Table A.1 [6] is evaluated the stiffness of the reinforcement steel.

The stiffness of the components is written in the following table, for negative bending moment.

<b>Beam-column joint, negative bending moment</b>				
<b>Component</b>	<b><math>F_{Rd,i}</math> [kN]</b>	<b><math>K_i</math> [kN/mm]</b>	<b><math>k_i=K_i/E</math> [mm]</b>	<b><math>z_i</math> [mm]</b>
<b>Reinforcement steel in concrete slab</b>	265,53	111	0,53	15
<b>Upper plate in tension</b>	2347,20	26427	126	116,5
<b>Vertical plate in tension</b>	396,40	3762	18	173
<b>Vertical plate in compression</b>	661,93	6486	31	295
<b>Lower plate in compression</b>	2347,20	26427	126	383,5

**Table 46.** Stiffness of the components of the joint for negative moment

Evaluation of the equivalent arm in agreement with EC3-1-8 6.3.3.1 (3):

$$Z_{mom-} = 243 \text{ mm}$$

$F_{Rd,comp}$ *	$F_{Rd,traz}$ *	$M_{Rd-} = Z_{mom-} \cdot \min(F_{Rd,comp}; F_{Rd,traz})$		$M_{Rd,b-}$ **
2979,36 kN	2979,36 kN	$M_{Rd+} = 243 \text{ mm} \cdot 2979,36 \text{ kN} = 724,44 \text{ kNm}$	>	478,15 kNm
* sum of the resistance of components				
** resistant plastic moment of the beam				

**Table 47.** Check of the hierarchy of resistance of the joint

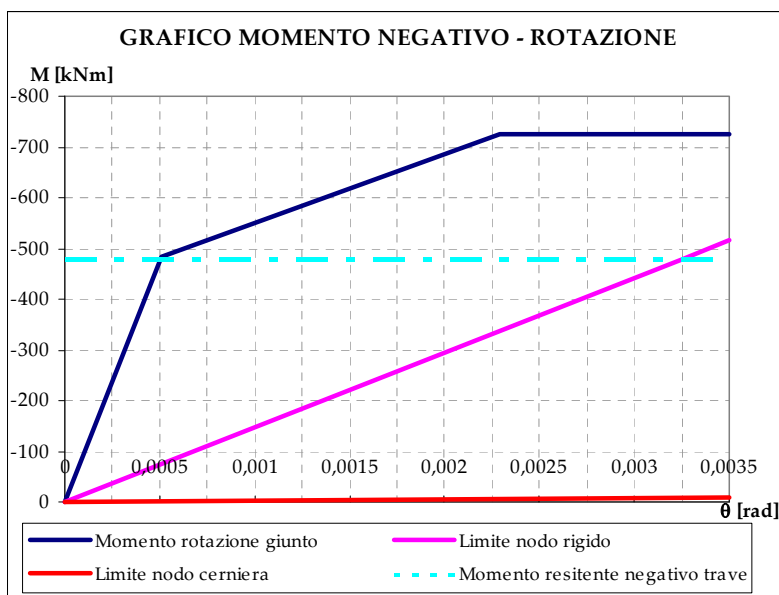
Evaluation of the equivalent stiffness of the springs:

<b>Formula</b>	<b>Stiffness</b>	<b><math>K_{eq}</math> [kN/mm]</b>	<b><math>k_{eq} = K_{eq}/E</math> [mm]</b>
$K_{eq} = \frac{\sum_{i=1}^n (k_i \cdot \delta_i^2)}{Z^2}$	<b>Spring in compression</b>	32692	156
	<b>Spring in tension</b>	31267	149

**Table 48.** Stiffness of the spring of the model joint

Initial joint stiffness:	$S_{j.in} = \frac{Z^2}{\sum_{i=1}^2 \left( \frac{1}{K_{eq,i}} \right)} = 945065 \text{ kNm}$
Secant stiffness:	$S_j = \frac{Z^2}{\mu \cdot \sum_{i=1}^2 \left( \frac{1}{K_{eq,i}} \right)} = \frac{Z^2}{(1,5)^{2,7} \cdot \sum_{i=1}^2 \left( \frac{1}{K_{eq,i}} \right)} = 316239 \text{ kNm}$
Limit stiffness	$S_{j.lim} = \frac{25 \cdot E \cdot I_{2,b}}{L_b} = \frac{25 \cdot 210000 \text{ MPa} \cdot 224644455 \text{ mm}^4}{8000 \text{ mm}} = 147423 \text{ kNm}$

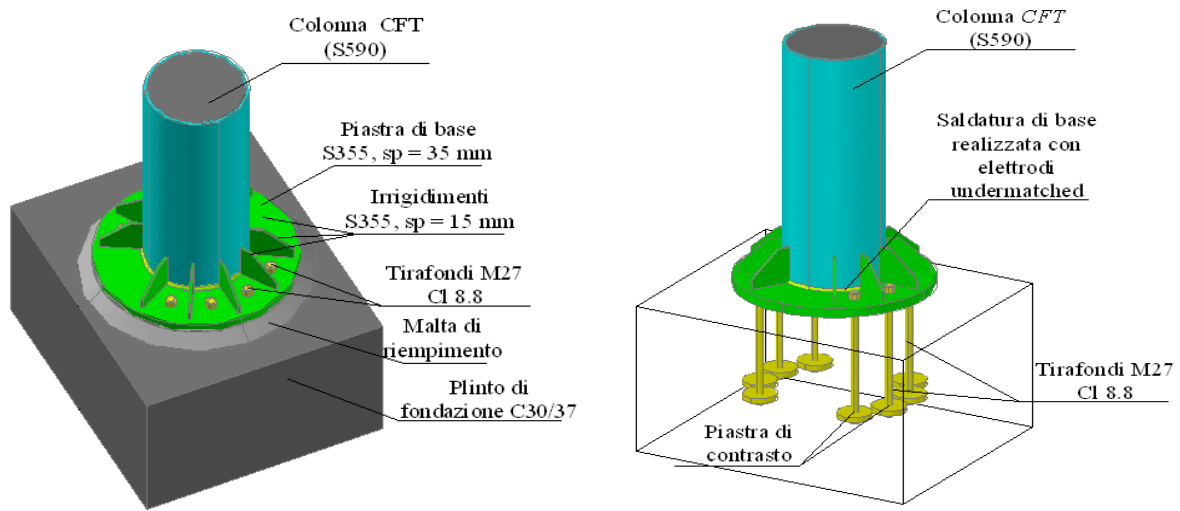
**Table 49.** Stiffness of the model joint



**Figure 31.** Moment rotation diagram of the joint for hogging bending moment.

### IV.3.6 Seismic design of the base joint

Three different base joints are analysing in ATTEL project. In this document only the design of typical base joint under seismic action is reported. The design of the joint is in agreement with EC8-1[7]. The geometry of the joint is shown in the following figure.



**Figure 32.** Geometry of the base joint.

#### IV.3.6.1 Geometry and action in the joint

Base plate: S355		Anchor bolts class 8.8					
D [mm]	t [mm]	n°	d [mm]	a [mm]	L <sub>t</sub> [mm]	D <sub>plate</sub> [mm]	t <sub>plate</sub> [mm]
760	35	8	27	300	600	150	25
				Distance between anchor bolt and foundation border	Length	Diameter of end plate	Thickness of end plate

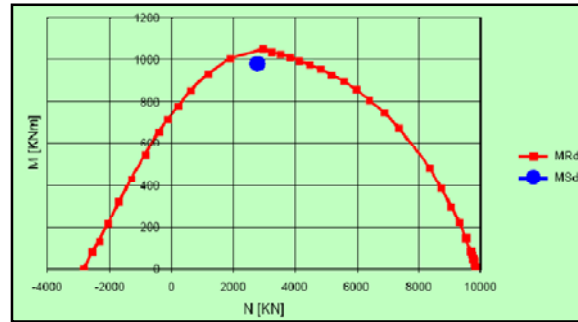
**Table 50.** Geometry of the base joint.

N <sub>F,max</sub> [kN]	M <sub>F,max</sub> [kNm]	V <sub>F,max</sub> [kN]
2784,79	983,78	442,34

**Table 51.** Actions in the base joint.

#### IV.3.6.2 Verification of base section under axial force and bending moment

$M_{F,max}$	$M_{Rd(NF,max)}$
983,78 kNm	1047 kN



**Figure 33.** M-N domain of the section of the base joint.

#### IV.3.6.3 Shear verification of base joint

Shear action on anchor bolt

$$F_{V,Ed} = \frac{V_{F,TOT}}{4} = \frac{442,34}{4} = 110,58 \text{ kN}$$

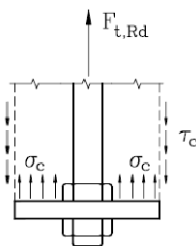
Shear resistance of an anchor bolt

$$F_{v,Rd} = \frac{\alpha_v f_{ub} A_{res}}{\gamma_{M2}} = \frac{0,6 \cdot 800 \text{ MPa} \cdot 459 \text{ mm}^2}{1,25} = 176,25 \text{ kN}$$

#### IV.3.6.4 Bearing verification of the base plate

$$F_{b,Rd} = \frac{k_1 \alpha_b f_u dt}{\gamma_{M2}} = \frac{2,5 \cdot 1 \cdot 510 \cdot 27 \cdot 35}{1,25} = 963,90 \text{ kN}$$

#### IV.3.6.5 Slip-off verification of the anchor bolt



Resistant of the anchor bolts

$$F_{T,Rd} = \frac{0,9 \cdot A_{res} \cdot f_u}{\gamma_{M2}} = 264,38 \text{ kN}$$

$$F_{R,sfil} = \frac{0,28 \sqrt{f_{cd}}}{(1 + d/a)} \pi d L_t + f_{cd} \left( 1 - \frac{D_{piastrina}}{2 \cdot a} \right) \pi \left( \frac{D_{piastrina}}{2} \right)^2 = 274,76 \text{ kN} > 264,3$$

#### IV.3.6.6 Flexural verification of the base plate

		$M_{Ed} = F_{T,Rd} \cdot \frac{L}{2} = 264,38 \cdot \frac{72,6}{2} = 9,65 \text{ kNm}$
		$M_{Rd} = \frac{f_y \cdot t^2 \cdot L_{diff}}{6 \cdot \gamma_{M0}} = \frac{355 \cdot 35^2 \cdot 145,3}{6 \cdot 1} = 10,51 \text{ kNm}$

#### IV.3.6.7 Verifications of the stiffeners

- section of the stiffener in tension.  $L_{diff}$  is the length of intersection between vertical stiffener and horizontal plate due to diffusion of the anchor bolt force .

$$N_{Rd,irrig} = \frac{t \cdot L_{diff} \cdot f_y}{\gamma_{M0}} = \frac{15 \cdot 145 \cdot 355}{1} = 772 \text{ kN} > F_{T,Rd} = 264,38 \text{ kN}$$

- horizontal weld between the vertical stiffener and the horizontal plate
- 

a	$L_{fillet}$	$n_{fillet}$	$f_{u,stiffners}$	$f_{u,plate}$	$F_{Ed, // weld}$	$F_{Ed, perp, weld}$	$M_{Ed, weld}$	$W_{el}$
4 mm	145 mm	2	510 MPa	510 MPa	0,00 kN	132,19 kN	0,00 kNm	14017 mm <sup>3</sup>

$\sigma_{\perp}$	$\tau_{//}$	$\tau_{\perp}$	$[\sigma_{\perp}^2 + 3 \cdot (\tau_{\perp}^2 + \tau_{//}^2)]^{0,5} < \frac{f_u}{\beta \cdot \gamma_{M2}}$	$\sigma_{\perp} < \frac{0,9 \cdot f_u}{\gamma_{M2}}$
161,16 MPa	0,00 MPa	161,16 MPa	<b>322,32 MPa &lt; 453,33 MPa</b>	<b>161,16 MPa &lt; 367,20 MPa</b>

**Table 52.** Verification of the horizontal welds.

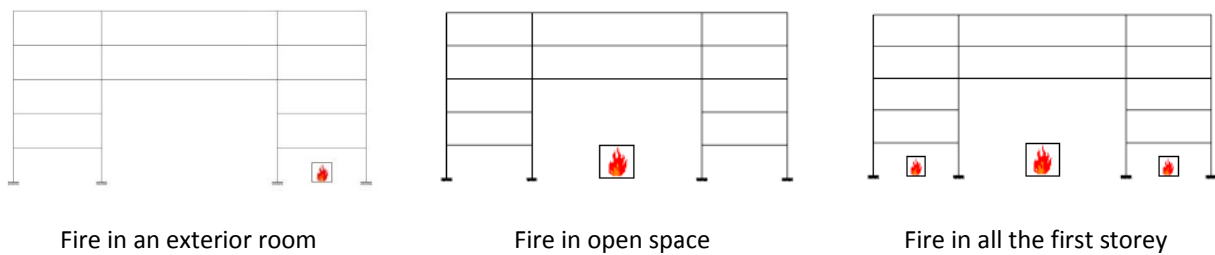
- vertical weld between the vertical stiffeners and the column. The anchor bolt generate the shear action and the bending moment on the vertical welds.

Shear	$V_{Ed} = F_{T,Rd} = 264,38 \text{ kN} < V_{Rd} = \frac{A_v f_y}{\gamma_{m0} \sqrt{3}} = \frac{15 \cdot 200 \cdot 355}{1 \sqrt{3}} = 614,88 \text{ kN}$
Bending Moment	$M_{Ed} = F_{T,Rd} b = 264,38 \cdot 112 = 29,61 \text{ kN} < M_{pl,Rd} = \frac{th^2 f_y}{6 \gamma_{M0}} = \frac{15 \cdot 200^2 \cdot 355}{6 \cdot 1} = 35,50 \text{ kNm}$

**Table 53.** Verification of the stiffeners.

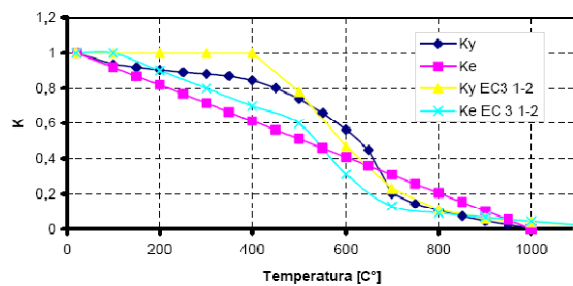


Three different fires scenario are considerate in the design.



**Figure 36.** Different fire scenario implemented in the design.

The difference between the steel S590 and S590\* is the resistance of the material with the temperature due to two different pairs of reduction factors for the strength and elasticity modulus. The reduction factors of the EC3 [5] are used for the steel S590 while factors found in literature are used for the steel S590\*



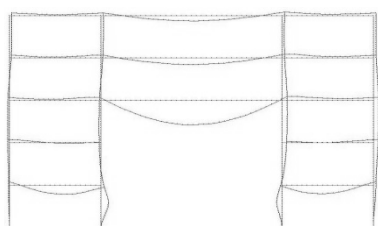
**Figure 37.** Reduction factors for the steel S590 and S590\*.

## V.1 Analysis of the frame with normalized ISO fire

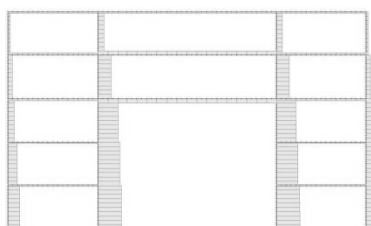
### V.1.1 CHS columns frame

The fire in all the first storey is the worst case. The results of the analysis are reported below.

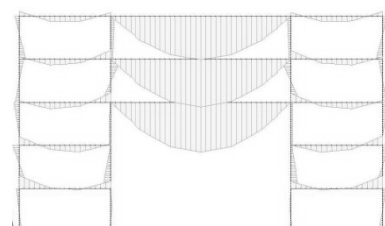
**Response of the building after 33 min.**



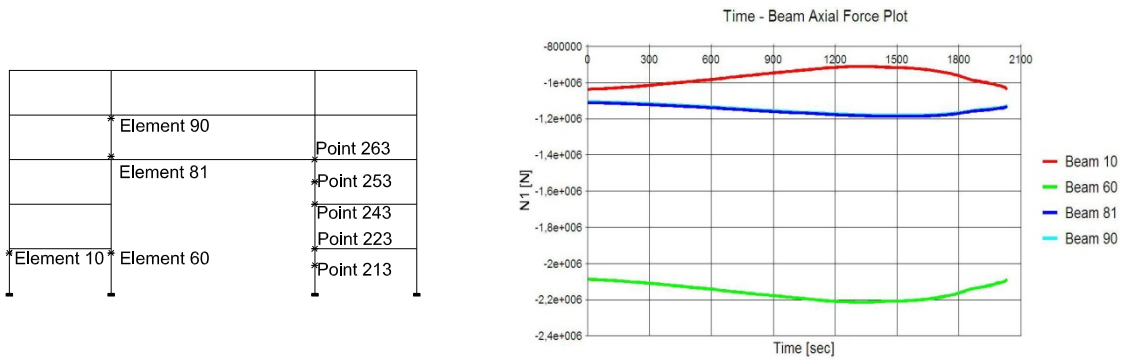
**Figure 38.** Shape of heat distortion



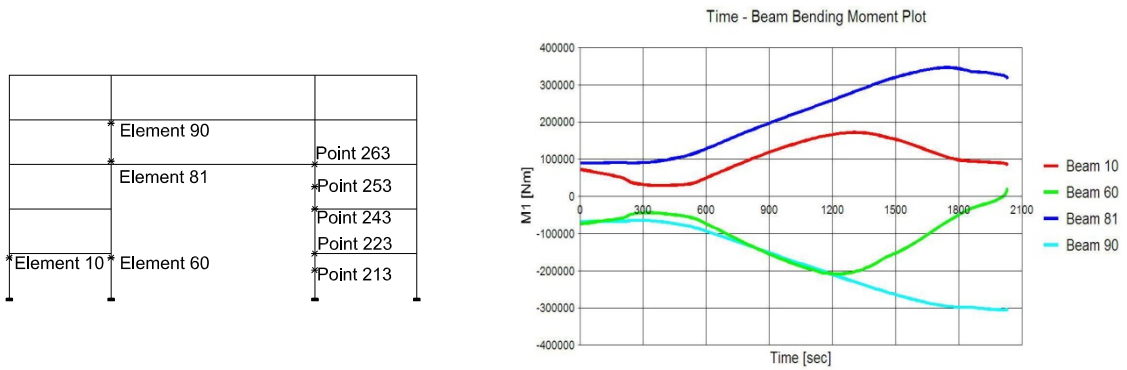
**Figure 39.** Axial force



**Figure 40.** Bending moment

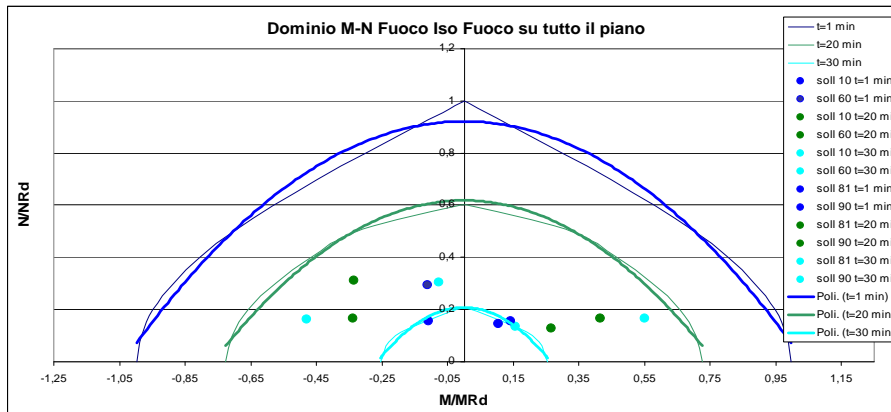


**Figure 41.** Curve time-axial force in the beams



**Figure 42.** Curve time-bending moment in the beams

The M-N domains by the FE program "SAFIR" for the different types of steel and the resistance times are showed below.



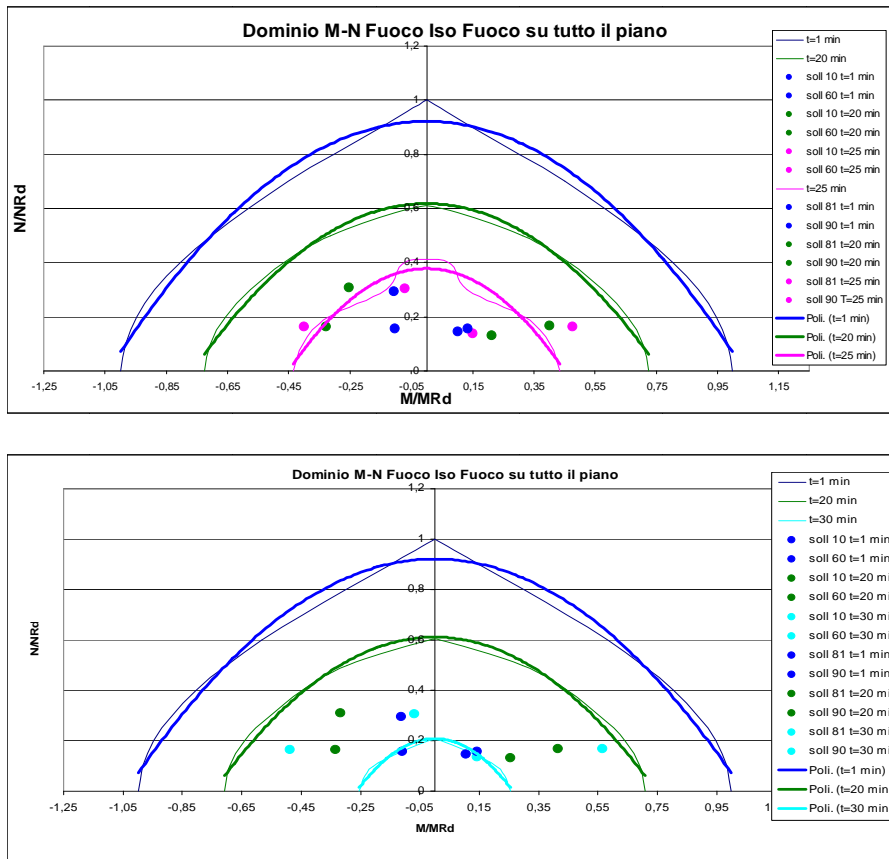


Figure 43. M-N domains for ISO fire in all the floor for S590, S355, S590\*.

ISO Fire in all first storey				ISO Fire in the open space			ISO Fire in lateral space		
Columns	CHS			CHS			CHS		
	S355	S590	S590*	S355	S590	S590*	S355	S590	S590*
R [min]	28	34	31	55	65	65	53	63	59

Table 55. Resistance times for different types of columns in different cases of fire.

### V.1.2 CFT columns frame

The fire in all the first storey is the worst case. The results of the analysis are reported below.

#### Response of the building after 60 min.

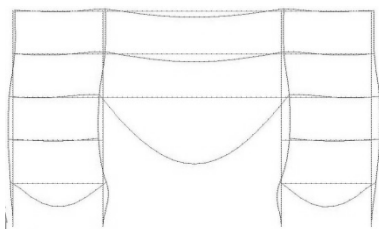


Figure 44. Shape of heat

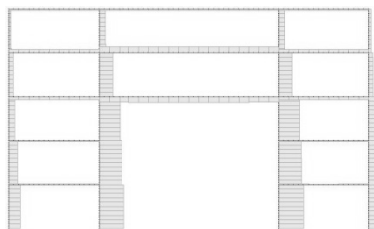


Figure 45. Axial force

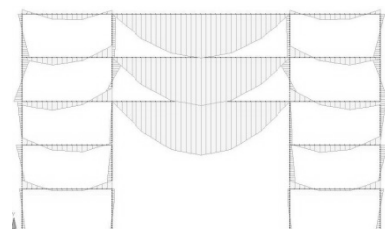


Figure 46. Bending moment

distortion

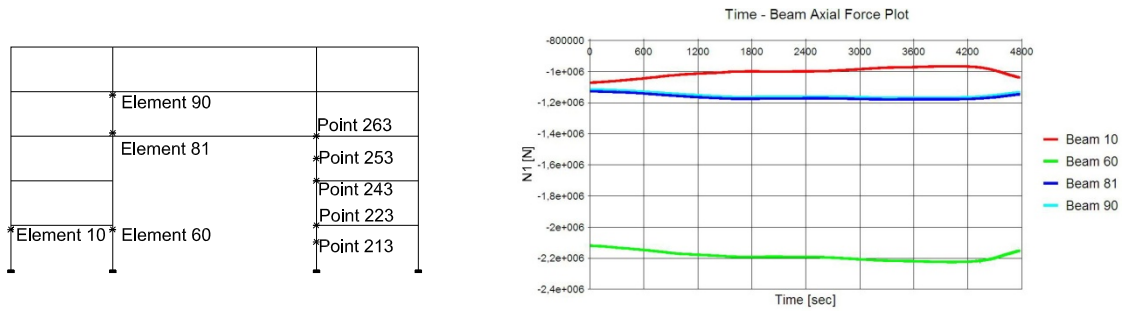


Figure 47. Curve time-axial force in the beams

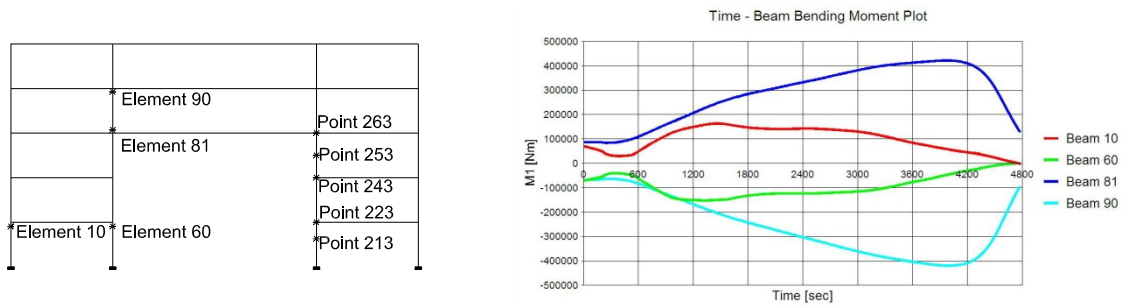


Figure 48. Curve time-bending moment in the beams

The M-N domains by the FE program "SAFIR" for the different types of steel and the resistance times are showed below.

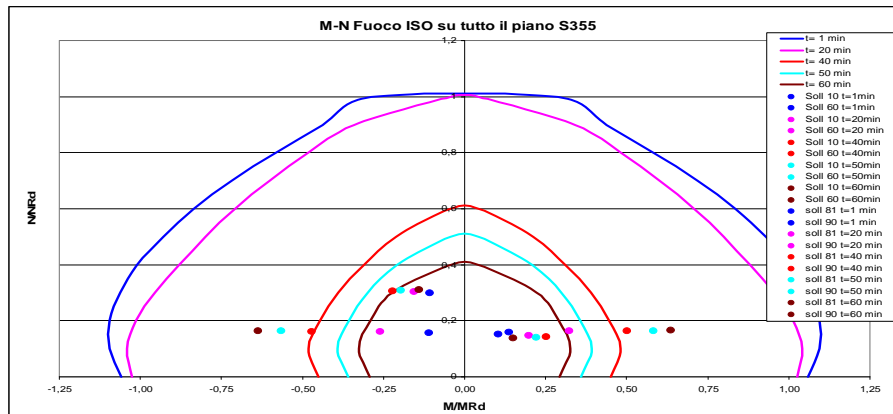


Figure 49. M-N domains for ISO fire in all the floor for S355, S590, S590\*.

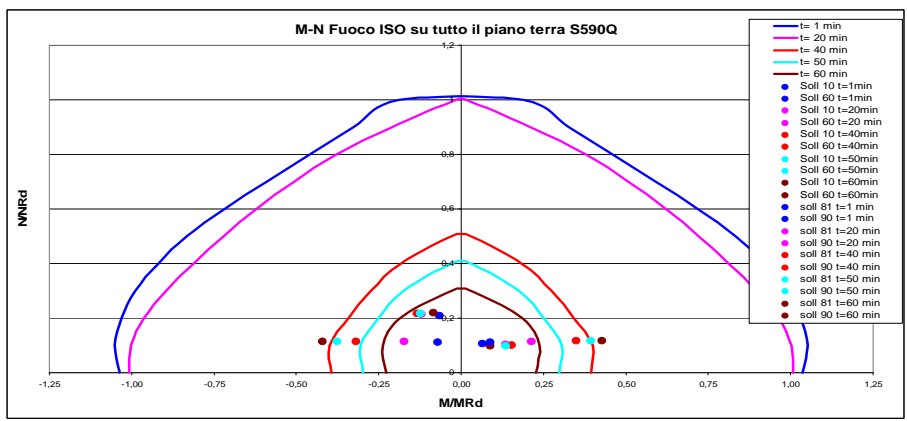
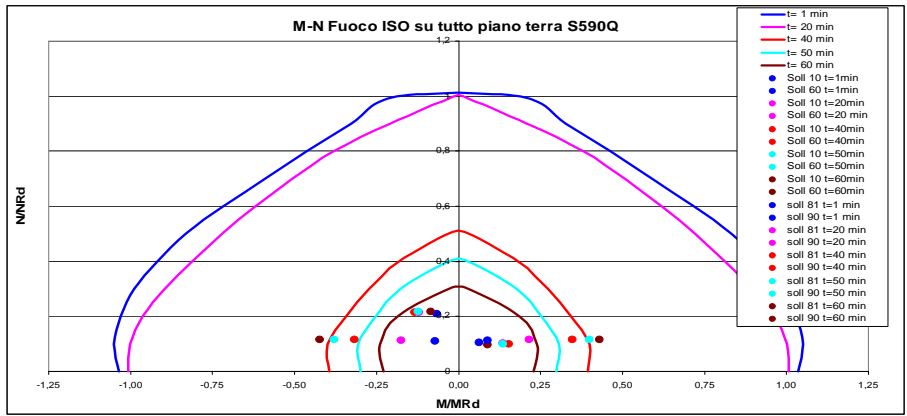


Figure 49. M-N domains for ISO fire in all the floor for S355, S590, S590\*.

Iso Fire in all first floor		Iso Fire in the open space			Iso Fire in lateral space				
Columns	CFT			CFT			CFT		
	S 355 C30/37	S 590 C30/37	S 590* C30/37	S 355 C30/37	S 590 C30/37	S 590* C30/37	S 355 C30/37	S 590 C30/37	S 590* C30/37
R [min]	79	79	80	100	100	100	91	91	90

Table 56. Resistance times for different types of columns in different cases of fire.

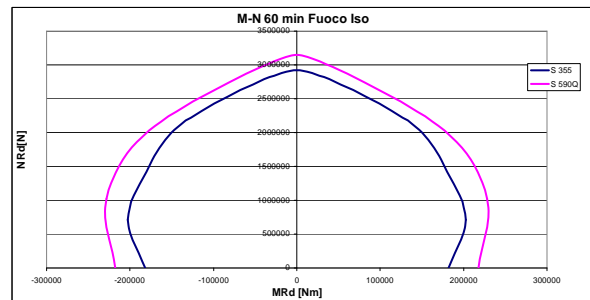
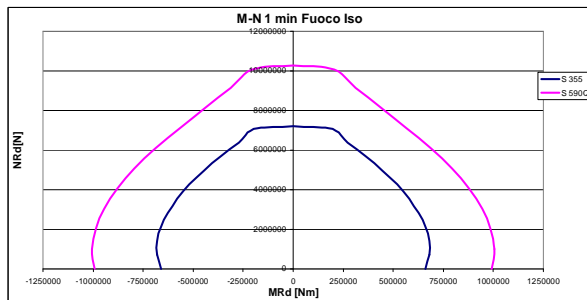


Figure 50. Comparison between M-N domains of S590 and S355 after 1min and 60 min.

## V.2 Analysis of the frame with natural fire

### V.2.1 CHS columns frame

The fire in all the first storey is the worst case. The results of the analysis are reported below.

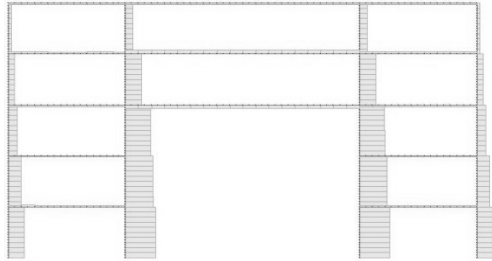


Figure 51. Axial force after 43 min

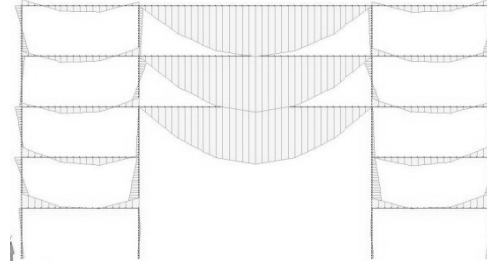


Figure 52. Bending moment after 43 min

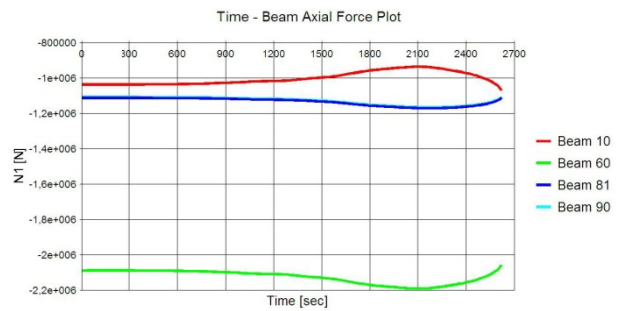
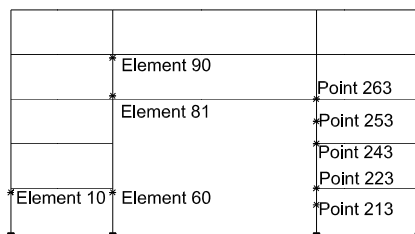


Figure 53. Curve time-axial force of the beams.

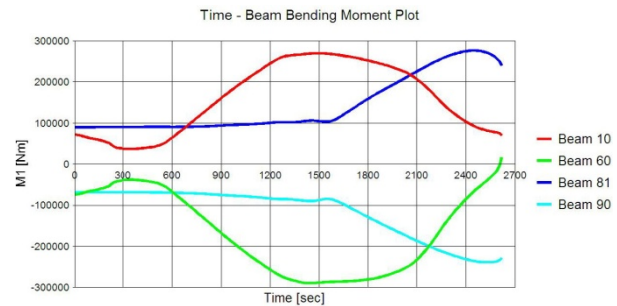
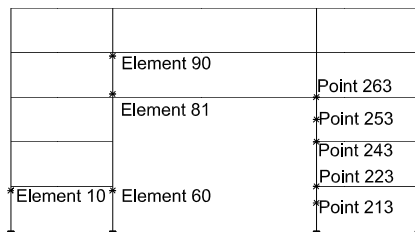


Figure 54. Curve time-bending moment of the beams.

The M-N domains by the FE program "SAFIR" for the different types of steel and the resistance times are showed below.

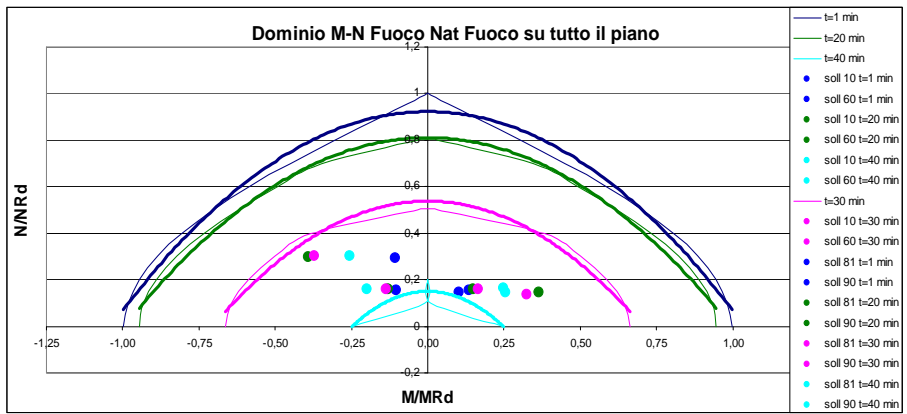


Figure 55. M-N domains for Natural fire in all the floor for S590, S355, S590\*.

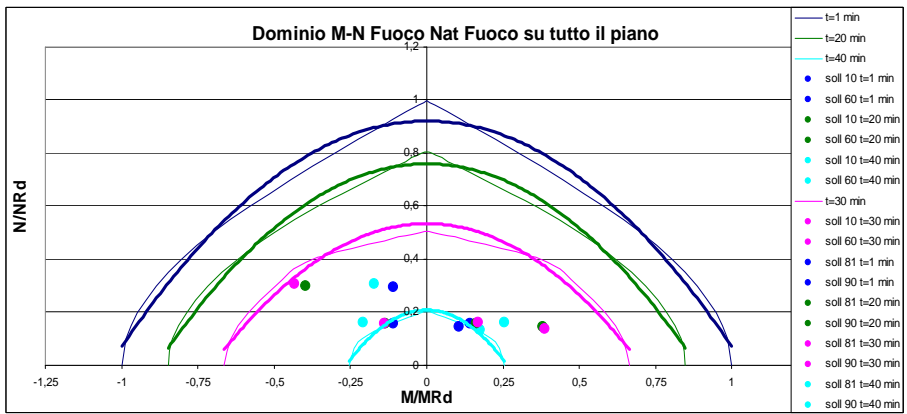
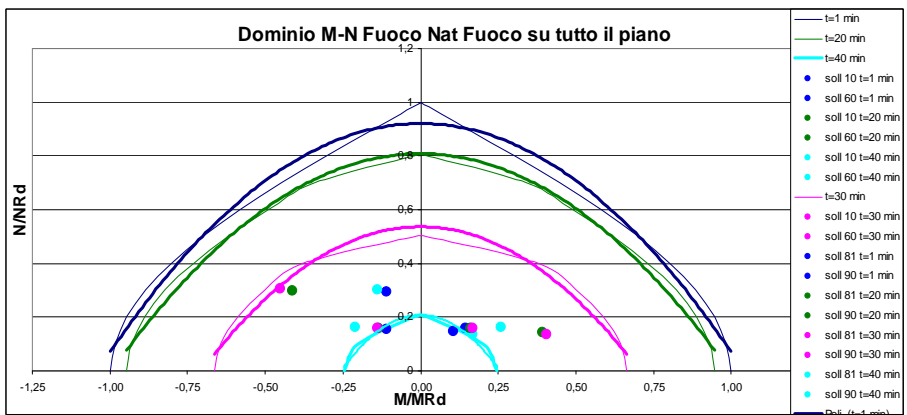


Figure 55. M-N domains for Natural fire in all the floor for S590, S355, S590\*.

Natural fire in all first floor			Natural fire in the open space			Natural fire in lateral space			
Columns	CHS			CHS			CHS		
	S355	S590	S590*	S355	S590	S590*	S355	S590	S590*
R [min]	38	43	42	55	65	65	53	59	57

Table 57. Resistance times for different kind of columns in different cases of fire.

## V.2.2 CFT columns frame

The fire in all the first storey is the worst case. The results of the analysis are reported below.

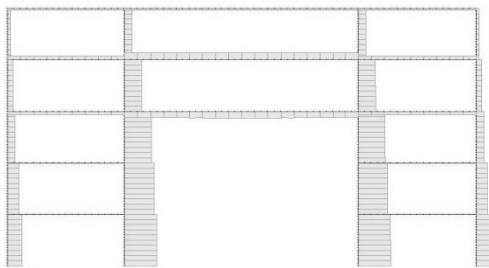


Figure 56. Axial force after 60 min

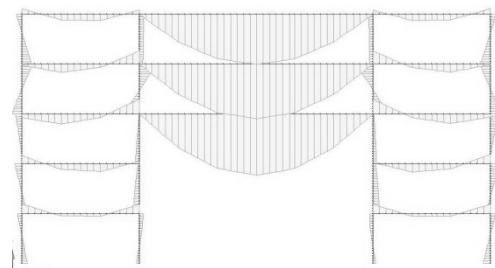


Figure 57. Bending moment after 60 min

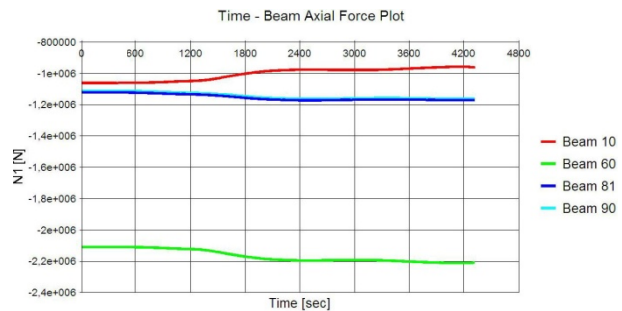
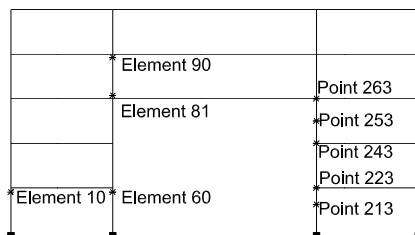


Figure 58. Curve time-axial force of the beams.

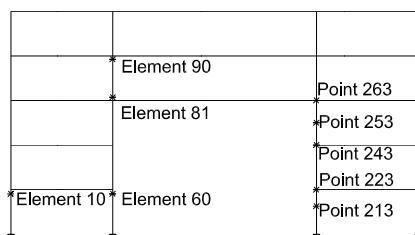


Figure 59. Curve time-bending moment of the beams.

The M-N domains by the FE program "SAFIR" for the different types of steel and the resistance times are showed below.

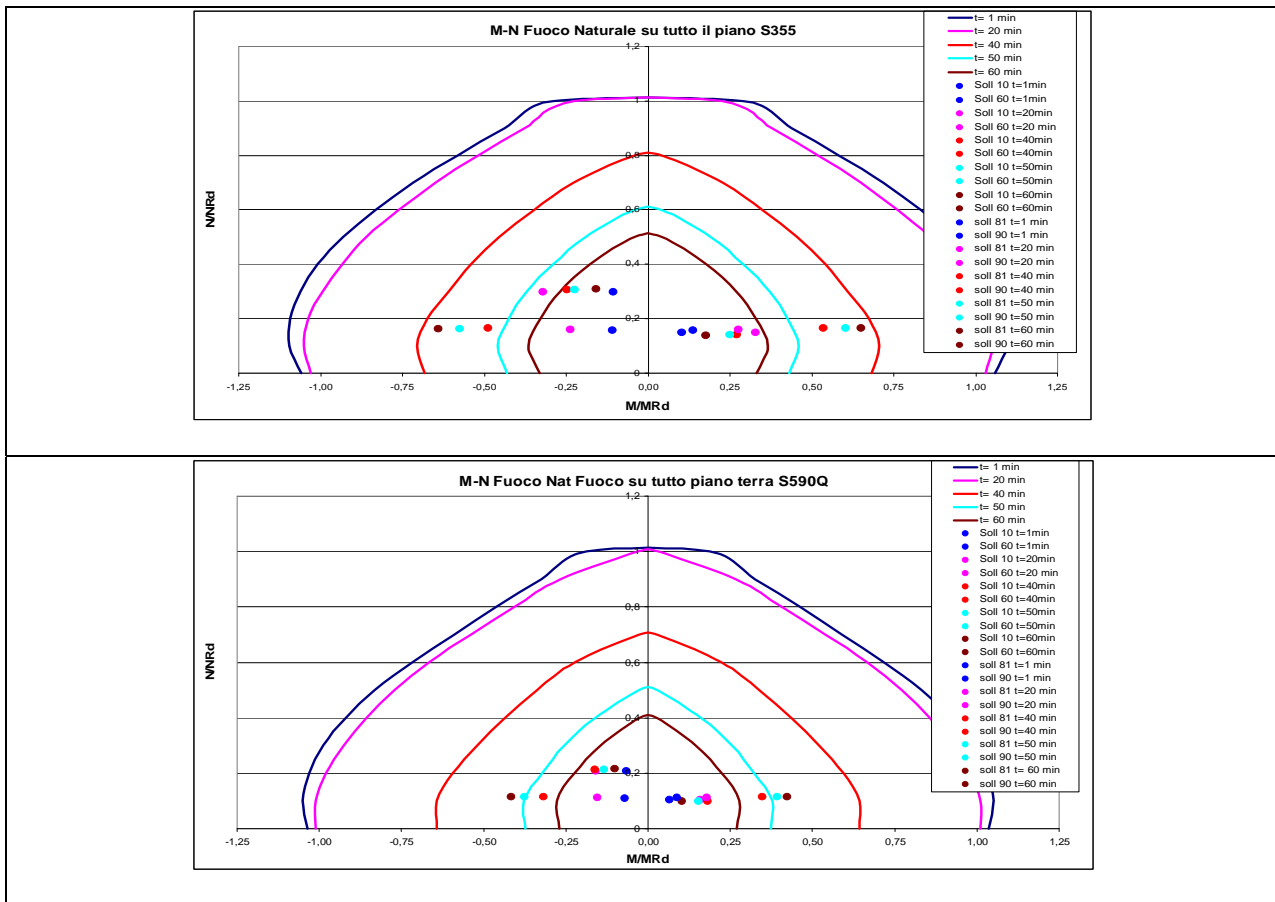


Figure 60. M-N domains for ISO fire in all the floor for S355, S590, S90\*.

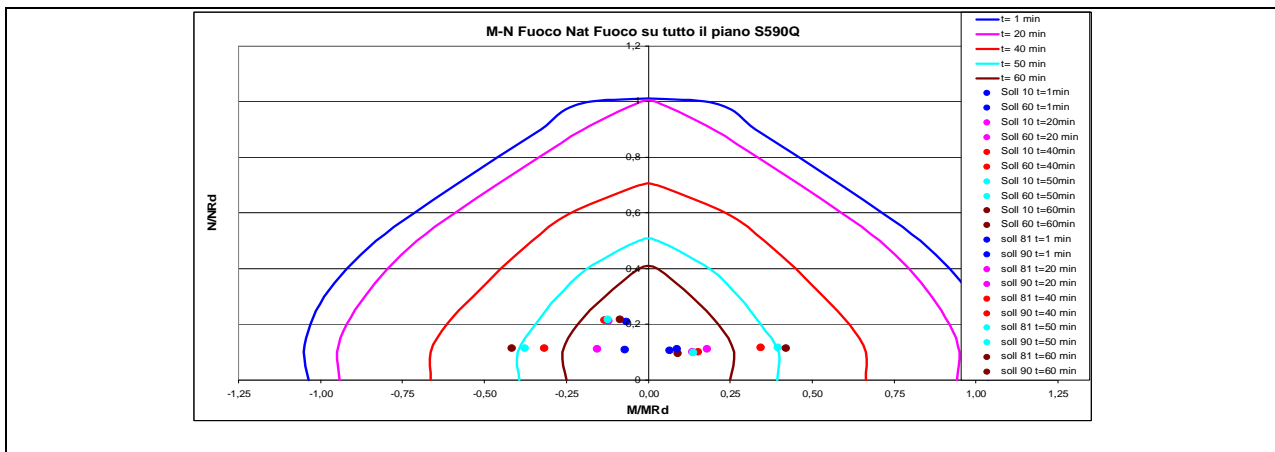
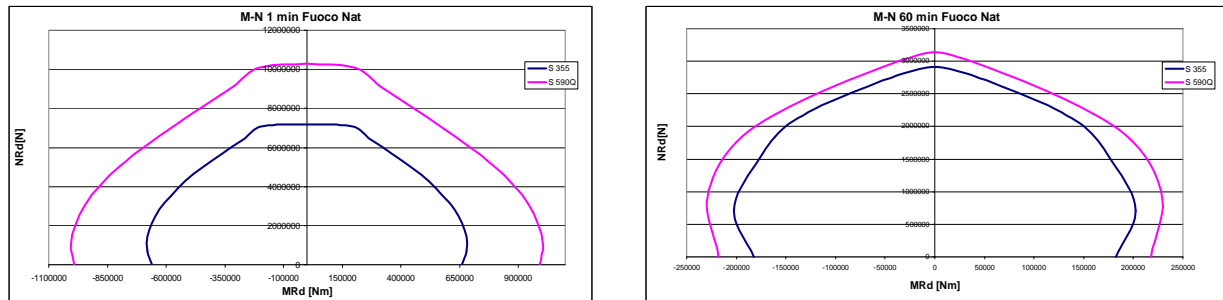


Figure 60. M-N domains for ISO fire in all the floor for S355, S590, S90\*.

Natural fire in all first floor			Natural fire in open space			Natural fire in lateral space			
Columns	CFT			CFT			CFT		
	S 355 C30/37	S 590 C30/37	S 590* C30/37	S 355 C30/37	S 590 C30/37	S 590* C30/37	S 355 C30/37	S 590 C30/37	S 590* C30/37
R [min]	91	91	91	100	100	100	92	90	92

**Table 58.** Resistance times for different kind of columns in different cases of fire.



**Figure 61.** Comparison between M-N domain of S590 and S355 after 1 min and 60 min.

## VI Conclusions

Static design both with CHS and CFT columns showed in all the verifications higher safety when the S590 steel is used instead of the S355 steel and the over-resistance is about 35% in the case of CHS columns and 40% for CFT columns.

The seismic design showed that:

- ✓ the verifications is not satisfied in the case of CHS and CFT columns in S355 steel grade;
- ✓ the verifications is satisfied for column with same geometry in S590 steel;
- ✓ the increase of cost for the use both of HSS material and of bigger columns in standard steel - in order to obtain the respect of the safety - is similar.

In fire design, considering the worst case with the fire in whole first storey, CHS columns in HSS steel grade ensure more resistance than the mild steel and less costs than protection in order to obtain the respect of the time of resistance (60 minutes).

In the case of CFT columns we have the same resistance in term of time between HSS and mild steel because the collapse is governed by concrete.

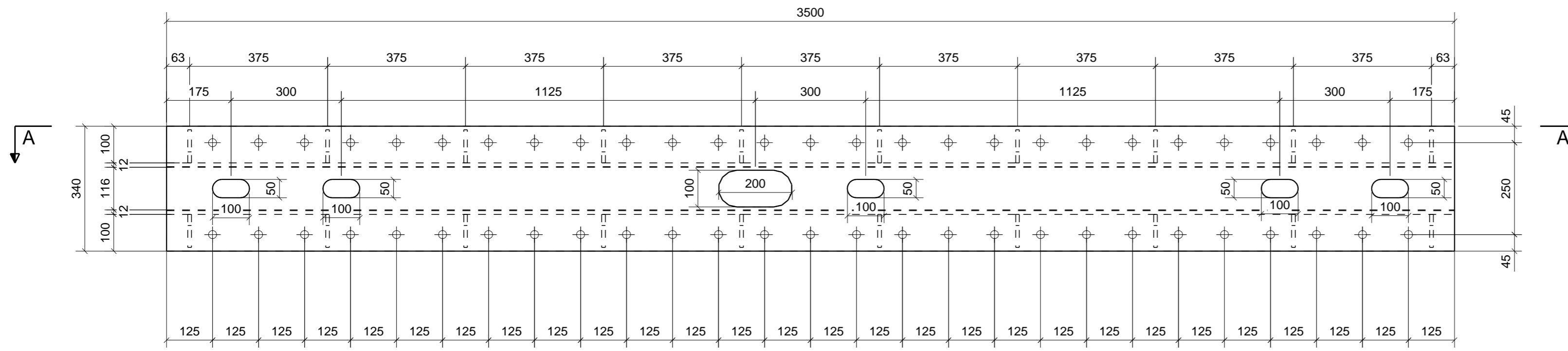
## VII References

- [1] ATTEL project: Performance-based approaches for high strength steel tubular columns and connections under earthquake and fire loading. Six-monthly Report 1, 2008.
- [2] UNI EN 1990 – “Bases of structural design”;
- [3] UNI EN 1991 – “Action of structures”, parts 1-1, 1-2, 1-3, 1-4, 1-6;
- [4] UNI EN 1992 – “Design of concrete structures”, parts 1-1, 1-2;
- [5] UNI EN 1993 – “Design of steel structures”, parts 1-1, 1-2, 1-3, 1-5, 1-8, 1-12;
- [6] UNI EN 1994 – “Design of composite steel and concrete structures”, parts 1-1, 1-2;

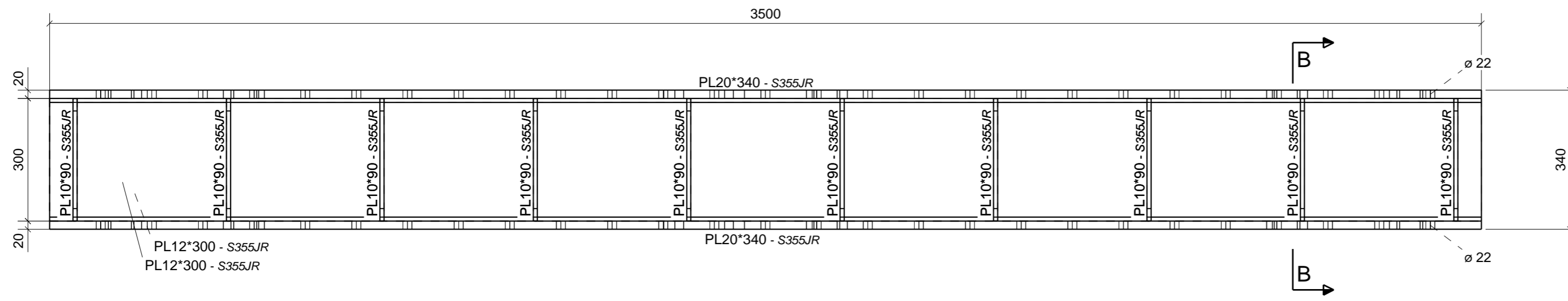
[7] UNI EN 1998 – “*Design of structures for earthquake resistance*”, parts 1, 5.

# **Appendix 4: Specimen drawings**

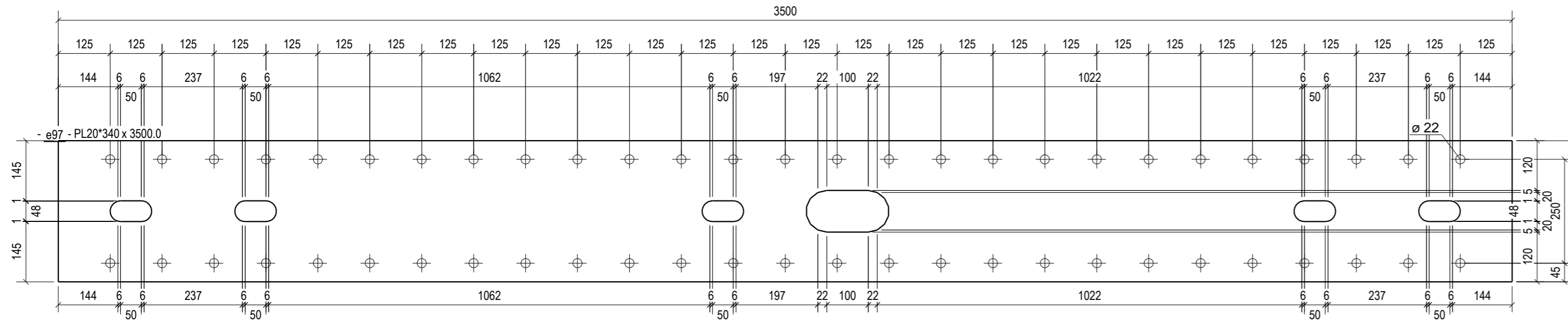
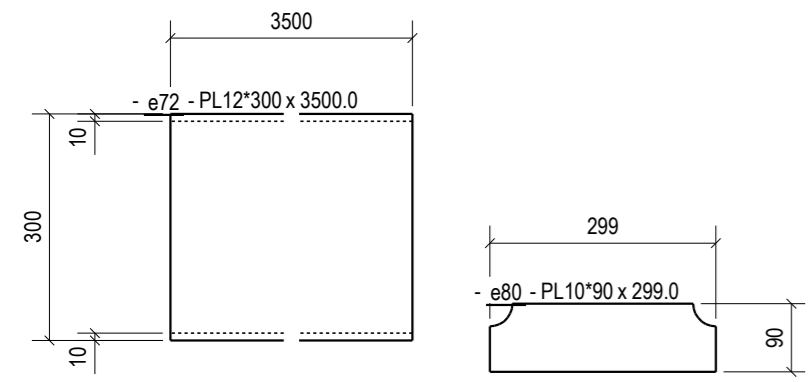
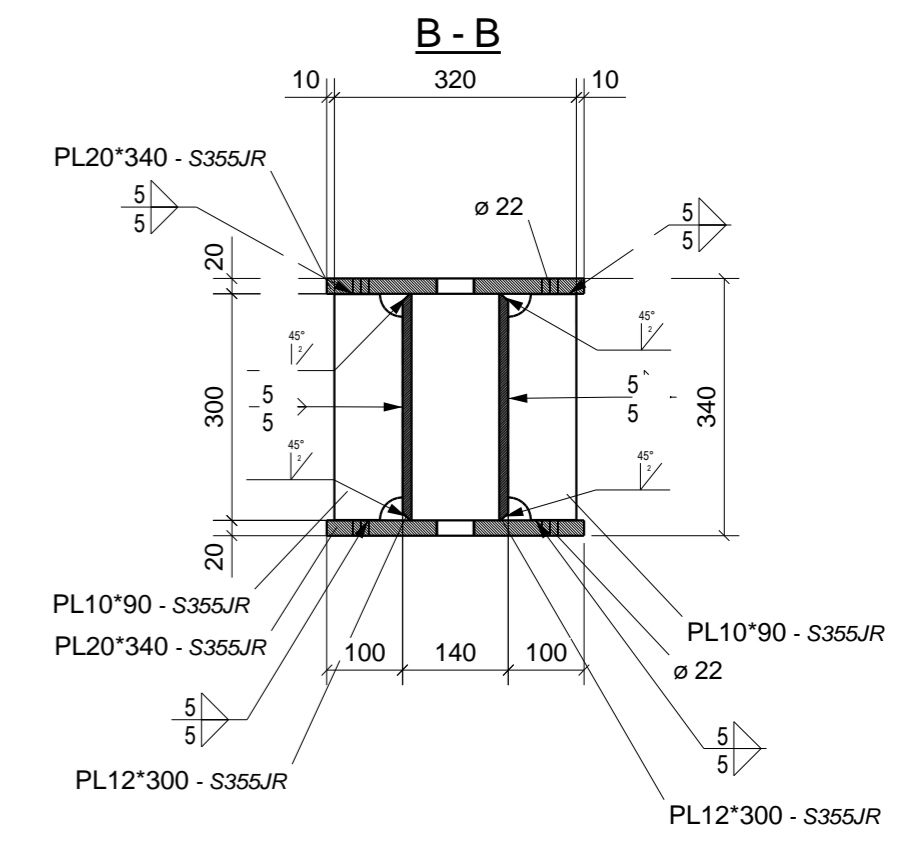
# 2 SPECIMENS



A - A



B - B



MARCA:	Descrizione:	Pezzi	Tratt.Sup.:			
UNITN24	PLATTE	2				
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
e72	PL12*300	S355JR	2	3500	2.1	96.2
e80	PL10*90	S355JR	20	299	0.1	2.0
e97	PL20*340	S355JR	2	3500	2.5	180.6
<b>Totale :</b>					<b>10.5</b>	<b>593.5</b>

ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE NAME - NOME DATUM - DATA INDEX

zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

**Standard Stahlbau Pichler**

Cliente: **UNIVERSITA TRENTO**  
Oggetto: **ATTEL**

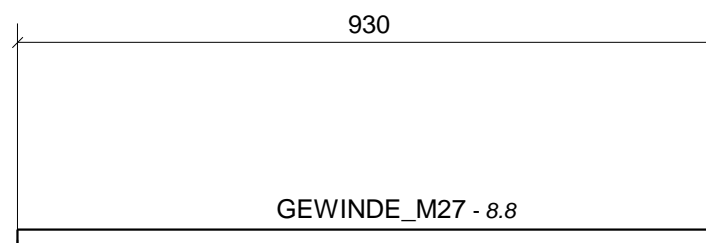
Comessa: **1559** Data: **01.09.2009**  
Reparto: **S** Scala: **1:10**

Gez./Dis.: **LUKAS H.** Geprüft / Controllato: **TB: Zelger** Datum/Data:  Geprüft / Controllato: **ST: Sommariva** Datum/Data:

**STAHLBAU PICHLER** GmbH - Srl  
Tel: 0471 / 065000 - Fax: 0471 / 065001  
E-MAIL: info@stahlbaupichler.com  
39100 BOZEN / BOLZANO

**UNITN24**

LOTTO	PEZZI
2	2

**16 SPECIMENS**

MARCA: <b>UNITN23</b>	Descrizione: GEWINDESTANGE	Pezzi 16	Tratt.Sup.:			
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
UNITN23	GEWINDE_M27	8.8	1	930	0.1	4.0
Totale :					0.1	4.0

revisione secondo indicazioni			LUKAS H.	28-09-2009	A
ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE			NAME - NOME	DATUM - DATA	INDEX

zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

Standard Stahlbau Pichler

Cliente: **UNIVERSITA TRENTO**

Oggetto: **ATTEL**

Comessa: **1559**

Data: **14.10.2009**

Reparto: **S**

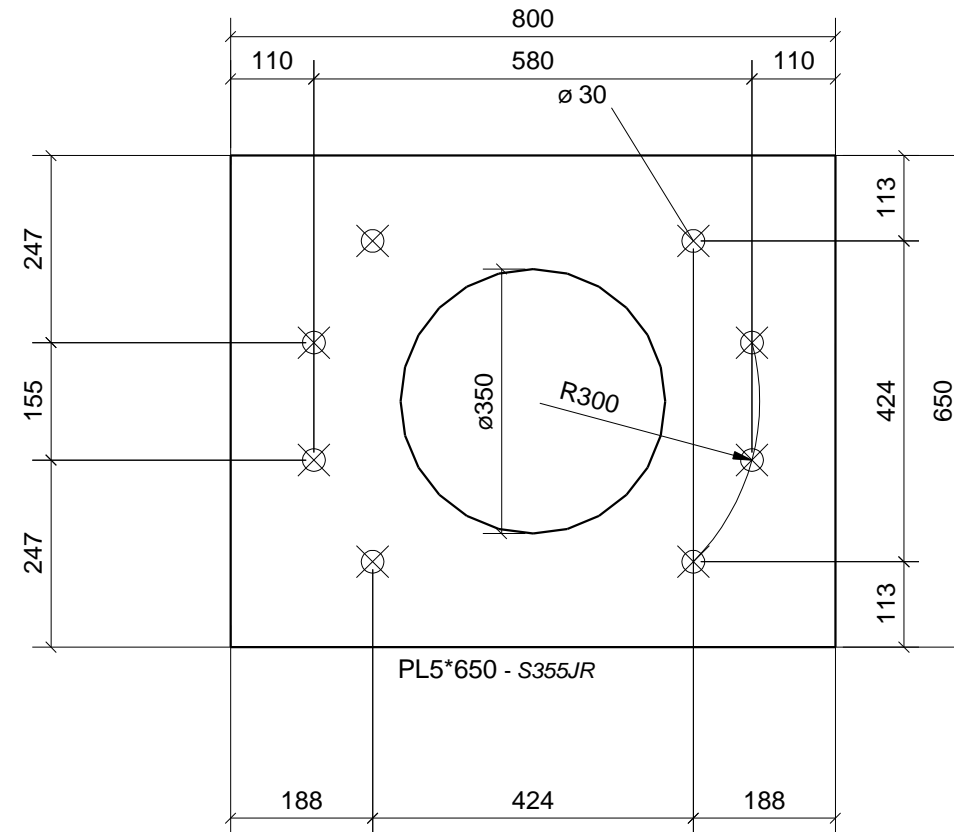
Scala: **1:10**



LOTTO	PEZZI
2	16

Gez./Dis.: LUKAS H.	Geprüft / Controllato: TB: Zelger	Datum/Data:	Geprüft / Controllato: ST: Sommariva	Datum/Data:
------------------------	--------------------------------------	-------------	---	-------------

## 2 SPECIMENS



MARCA: <b>UNITN22</b>	Descrizione: PLATTE	Pezzi 2	Tratt.Sup.:			
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
UNITN22	PL5*650	S355JR	1	800	0.9	16.7
Totale :					0.9	16.7

ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE      NAME - NOME      DATUM - DATA      INDEX

zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

**Standard Stahlbau Pichler**

Cliente: **UNIVERSITA TRENTO**

Oggetto: **ATTEL**

Comessa: **1559**

Data: **01.09.2009**

Reparto: **S**

Scala: **1:10**



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39100 BOZEN/BOLZANO



Disegno:  
**UNITN22**

Gez./Dis.:  
**LUKAS H.**

Geprüft / Controllato:  
TB: **Zelger**

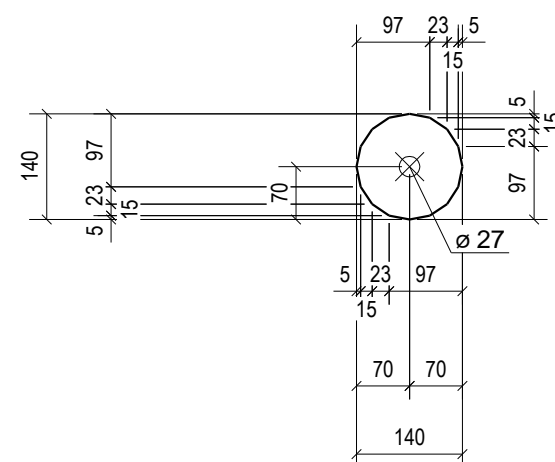
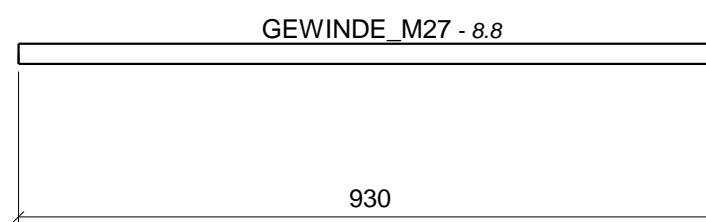
Datum/Data:

Geprüft / Controllato:  
ST: **Sommariva**

Datum/Data:

LOTTO	PEZZI
2	2



# 8 SPECIMENS



MARCA: <b>UNITN21</b>	Descrizione: <b>GEWINDESTANGE</b>	Pezzi <b>8</b>	Tratt.Sup.:			
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
MARCA: <b>UNITN25</b>	Descrizione: <b>PLATTE</b>	Pezzi <b>32</b>	Tratt.Sup.:			
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
UNITN21	GEWINDE_M27	8.8	1	930	0.1	4.0
UNITN25	PL25*140	S355JR	1	140	0.0	2.9
Totale :					0.1	6.9

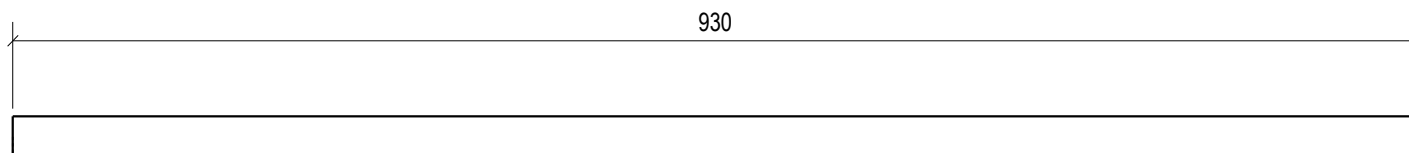
revisione secondo indicazioni	LUKAS H.	28-09-2009	A
ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE	NAME - NOME	DATUM - DATA	INDEX

zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature : Standard Stahlbau Pichler

Cliente: <b>UNIVERSITA TRENTO</b>		
Oggetto: <b>ATTEL</b>		
Comessa: <b>1559</b>	Data: <b>24.09.2009</b>	
Reparto: <b>S</b>	Scala: <b>1:10</b>	
Gez./Dis.: <b>LUKAS H.</b>	Geprüft / Controllato: <b>TB: Zelger</b>	Datum/Data:
	Geprüft / Controllato: <b>ST: Sommariva</b>	Datum/Data:

LOTTO	PEZZI
2	40

# 8 SPECIMENS



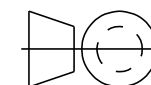
LOTTO	PEZZI
2	8

Quantità	per Marca
8	UNITN21

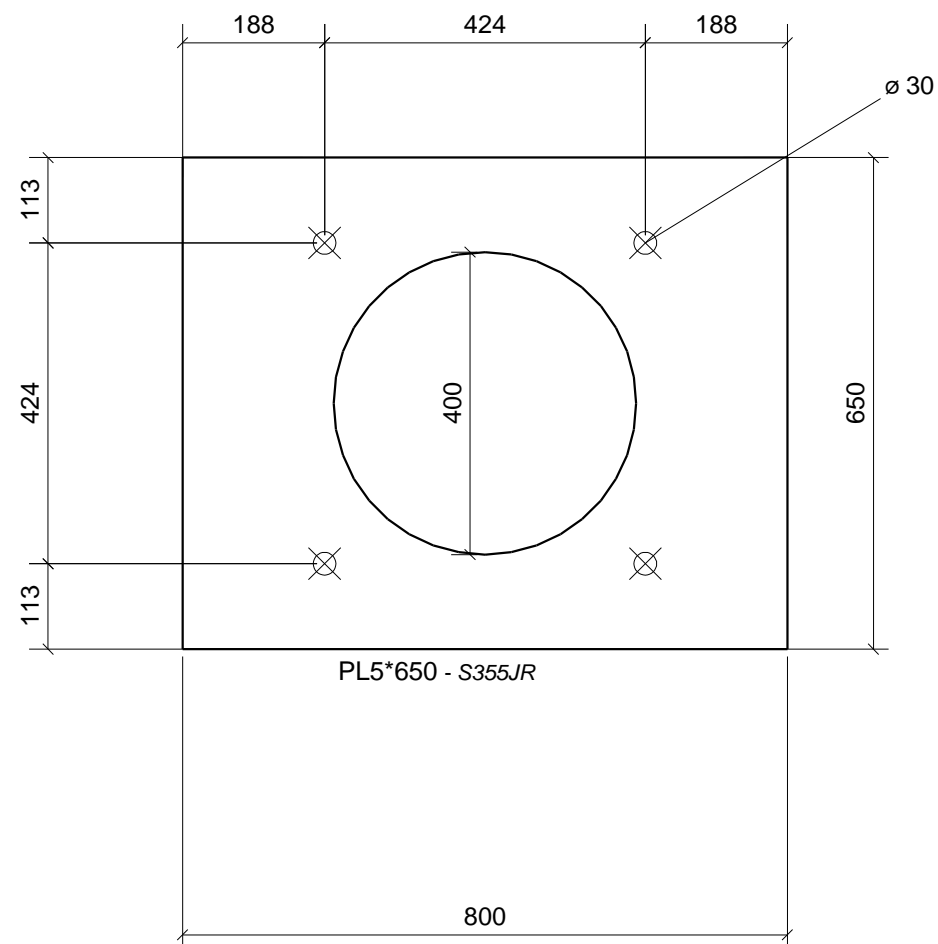
Cliente: UNIVERSITA TRENTO	Comessa: 1559	Reparto: S	Disegno: GEWINDESTAN
Descrizione: GEWINDESTANGE	Profilo: GEWINDE M27		Quantità: 8
Lunghezza: 930	Posizione: UNITN21		Peso: 4.1
Superficie: 0.08			Data: 13.11.2009
Materiale: 8.8			Modifica: 28-09-2009 Rev: A




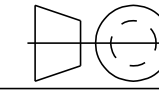
**STAHLBAU  
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Tel.0471/065000 - Fax : 0471/065001  
E-MAIL: info@stahlbaupichler.com  
39100 BOZEN/BOLZANO



**2 SPECIMENS**



MARCA: <b>UNITN20</b>	Descrizione: PLATTE	Pezzi 2	Tratt.Sup.:			
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
UNITN20	PL5*650	S355JR	1	800	0.8	15.5
Totale :					0.8	15.5

ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE		NAME - NOME		DATUM - DATA		INDEX
zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte : scostamenti ammessi per misure senza tolleranze / saldature :				<b>Standard Stahlbau Pichler</b>		
Cliente:	<b>UNIVERSITA TRENTO</b>		 <b>STAHLBAU PICHLER</b> GmbH-Srl <small>STÄHLBAU PICHLER</small>		Disegno: <b>UNITN20</b>	
Oggetto:	<b>ATTEL</b>					
Comessa:	<b>1559</b>	Data:	<b>01.09.2009</b>			
Reparto:	<b>S</b>	Scala:	<b>1:10</b>			
Gez./Dis.:	<b>LUKAS H.</b>	Geprüft / Controllato:	<b>TB: Zelger</b>	Datum/Data:	<b>ST: Sommariva</b>	Datum/Data:

LOTTO	PEZZI
2	2

# 8 SPECIMENS


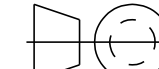


MARCA: <b>UNITN19</b>	Descrizione: <b>GEWINDESTANGE</b>	Pezzi <b>8</b>	Tratt.Sup.:			
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
UNITN19	GEWINDE_M30	10.9	1	885	0.1	4.7
Totale :					0.1	4.7

revisione secondo indicazioni			LUKAS H.	28-09-2009	A
ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE			NAME - NOME	DATUM - DATA	INDEX

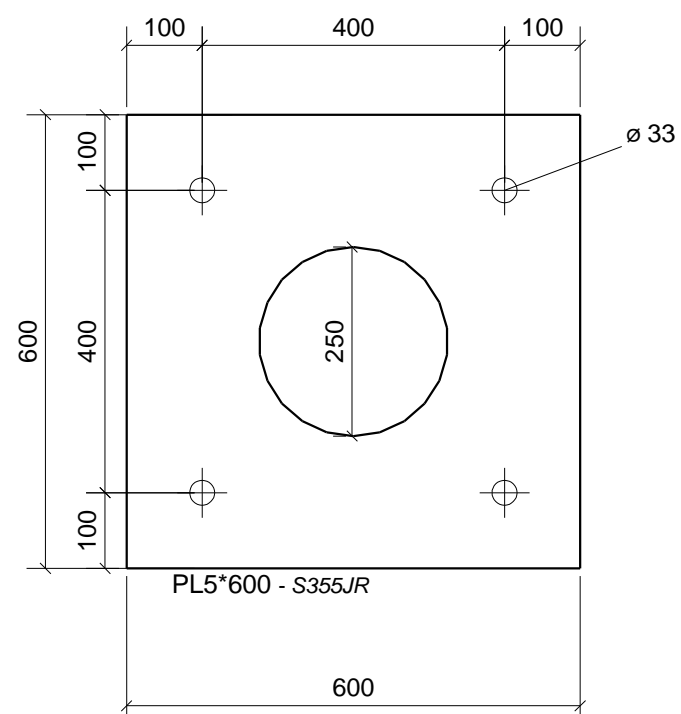
zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

**Standard Stahlbau Pichler**

Cliente: <b>UNIVERSITA TRENTO</b>	 <p>STAHLBAU PICHLER GmbH-Srl Tel.0471/065000 - Fax : 0471/065001 E-MAIL: info@stahlbaupichler.com 39100 BOZEN/BOLZANO</p>	 <p>Disegno: <b>UNITN19</b></p>
Oggetto: <b>ATTEL</b>		
Comessa: <b>1559</b>	Data: <b>14.10.2009</b>	
Reparto:	Scala: <b>1:10</b>	
Gez./Dis.:	Geprüft / Controllato: TB: Zelger	Datum/Data:
	Geprüft / Controllato: ST: Sommariva	Datum/Data:

LOTTO	PEZZI
2	8

# 2 SPECIMENS



MARCA: <b>UNITN18</b>	Descrizione: PLATTE	Pezzi 2	Tratt.Sup.:			
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
UNITN18	PL5*600	S355JR	1	600	0.6	12.2
Totale :					0.6	12.2

ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE      NAME - NOME      DATUM - DATA      INDEX

zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

Standard Stahlbau Pichler

Cliente: **UNIVERSITA TRENTO**

Oggetto: **ATTEL**

Comessa: **1559**

Data: **01.09.2009**

Reparto: **S**

Scala: **1:10**

Disegno:  
**UNITN18**

Gez./Dis.:  
**LUKAS H.**

Geprüft / Controllato:  
TB: **Zelger**

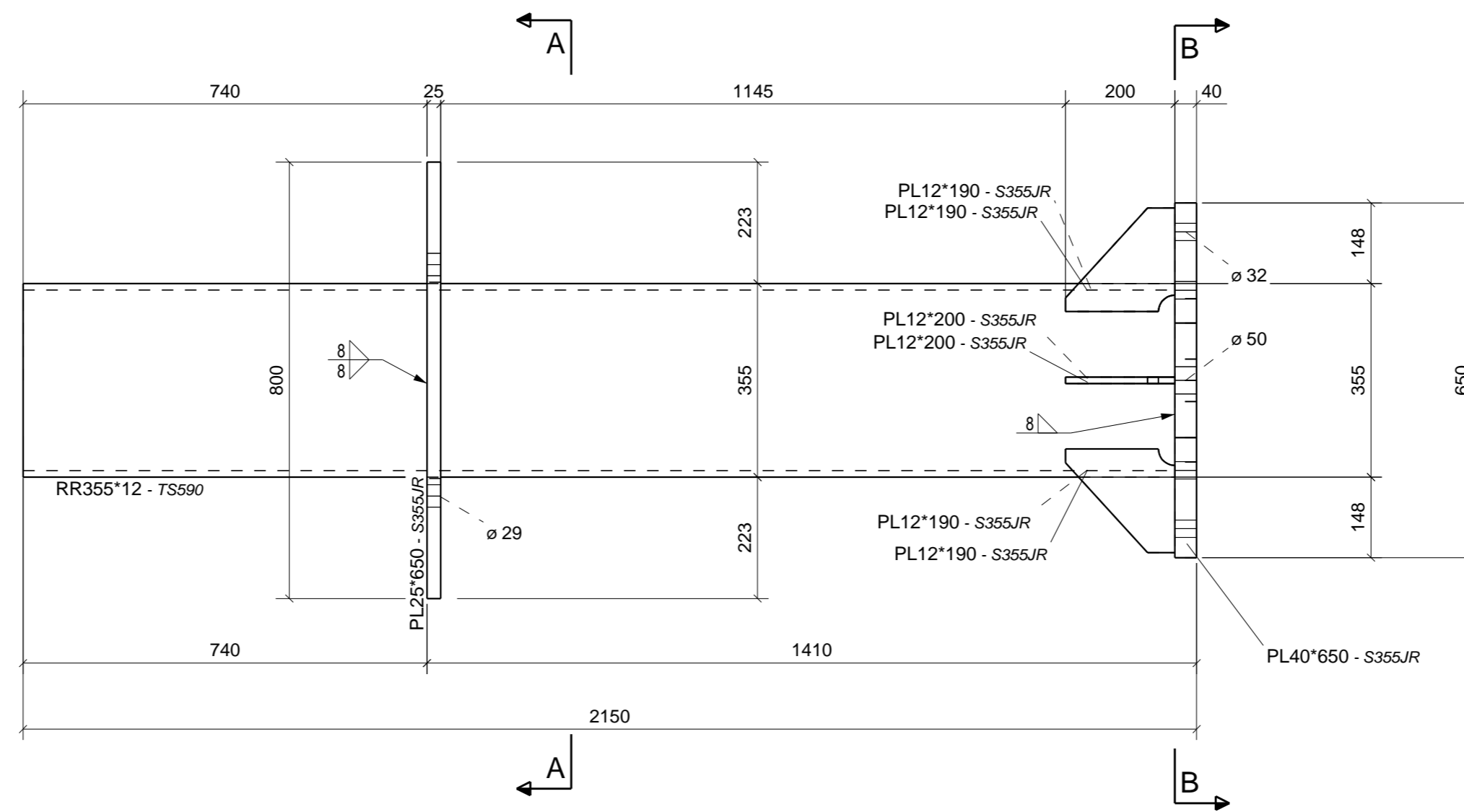
Datum/Data:

Geprüft / Controllato:  
ST: **Sommariva**

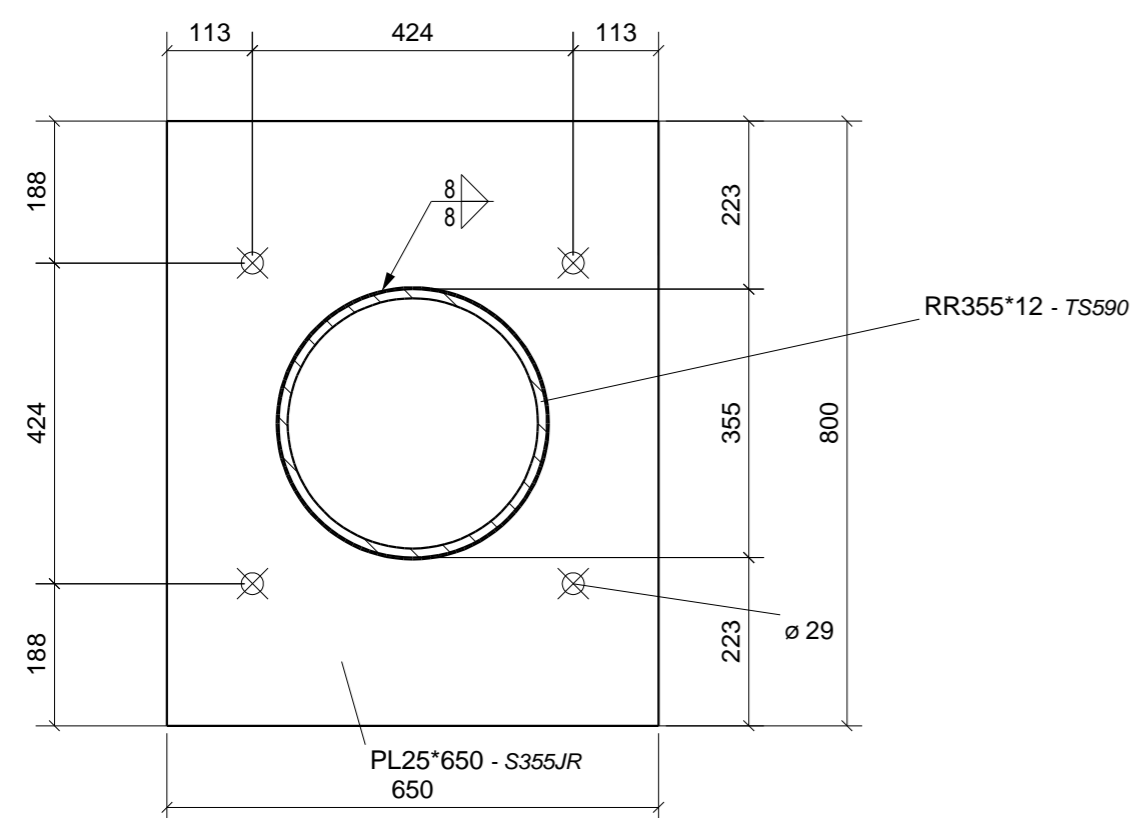
Datum/Data:

LOTTO	PEZZI
2	2

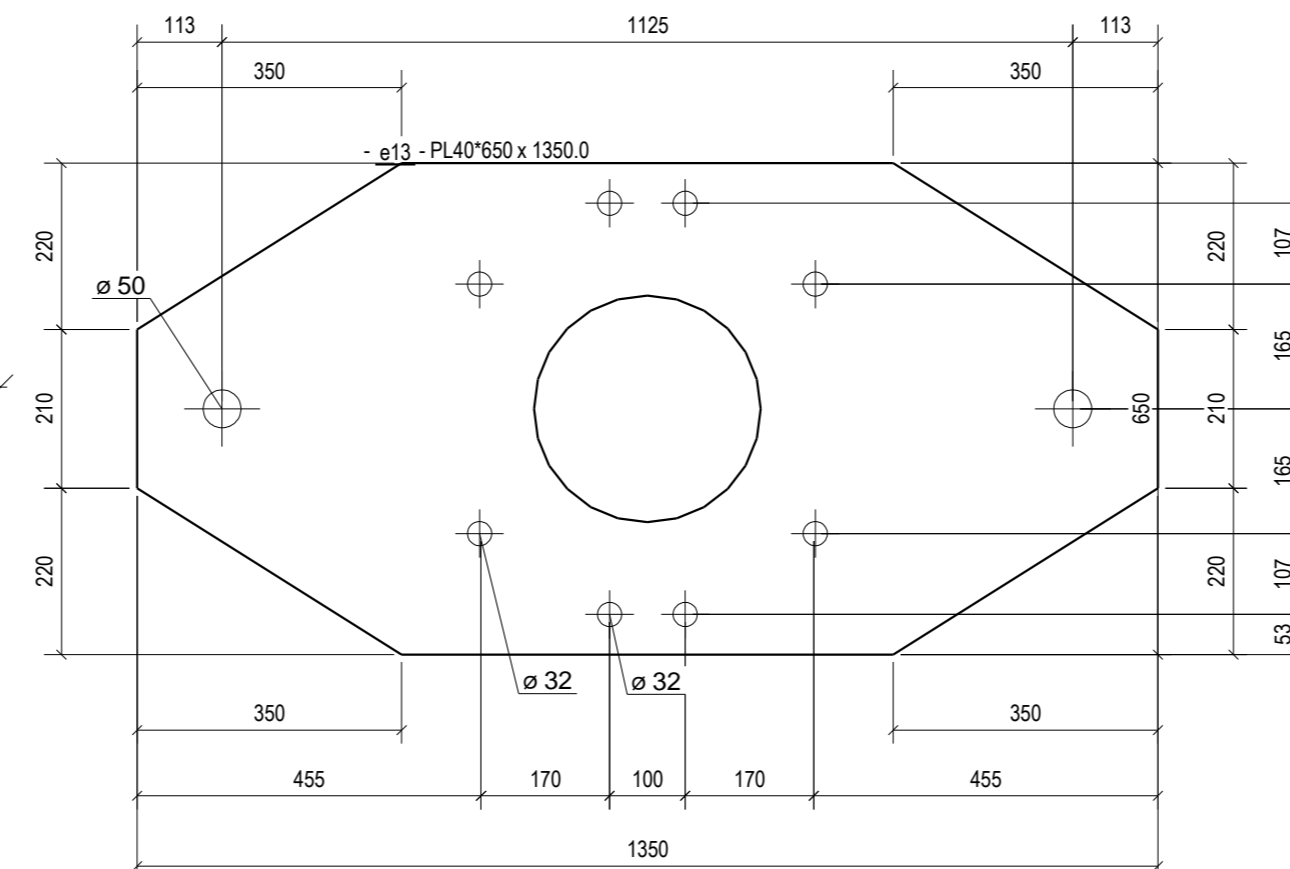
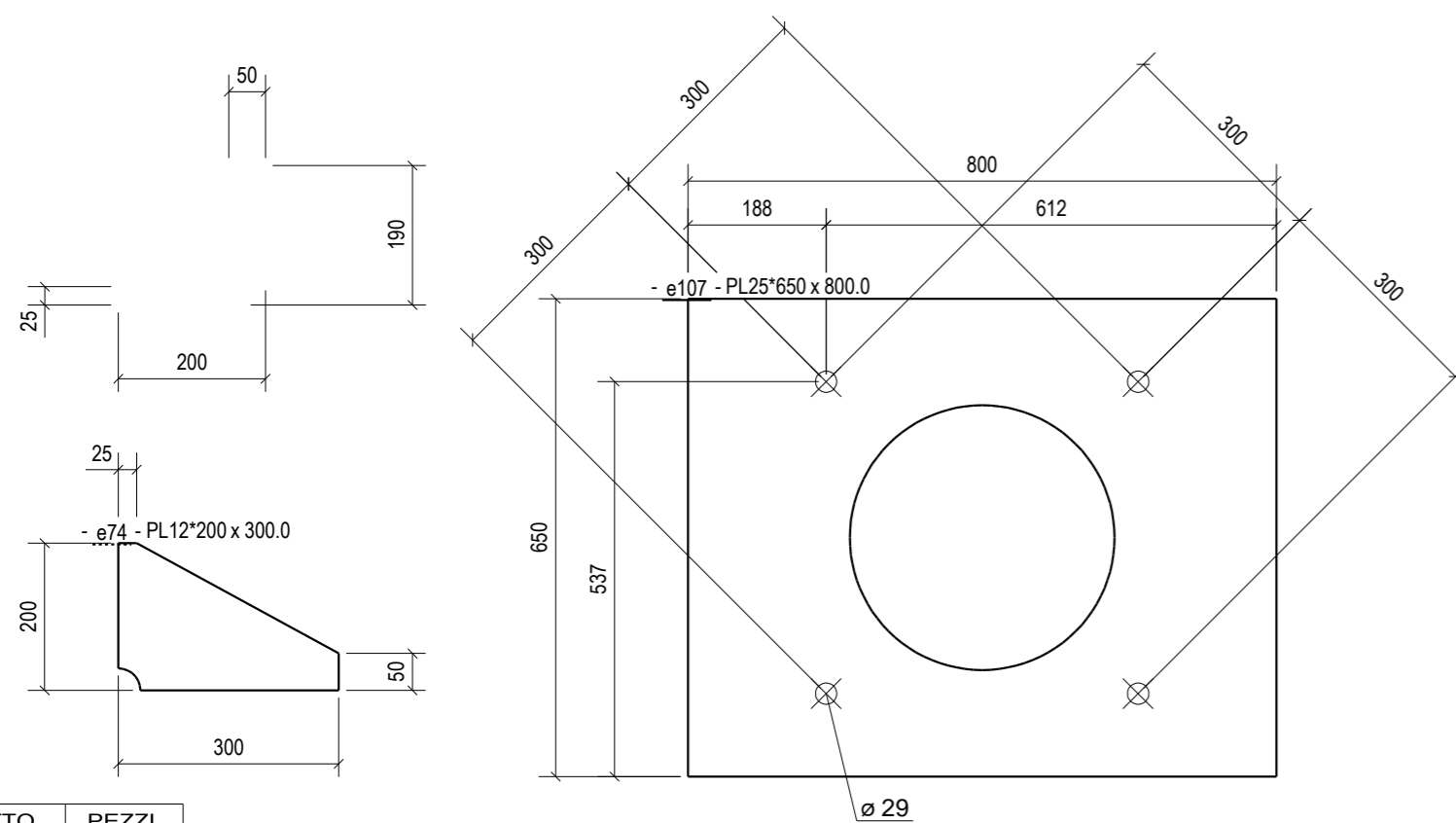
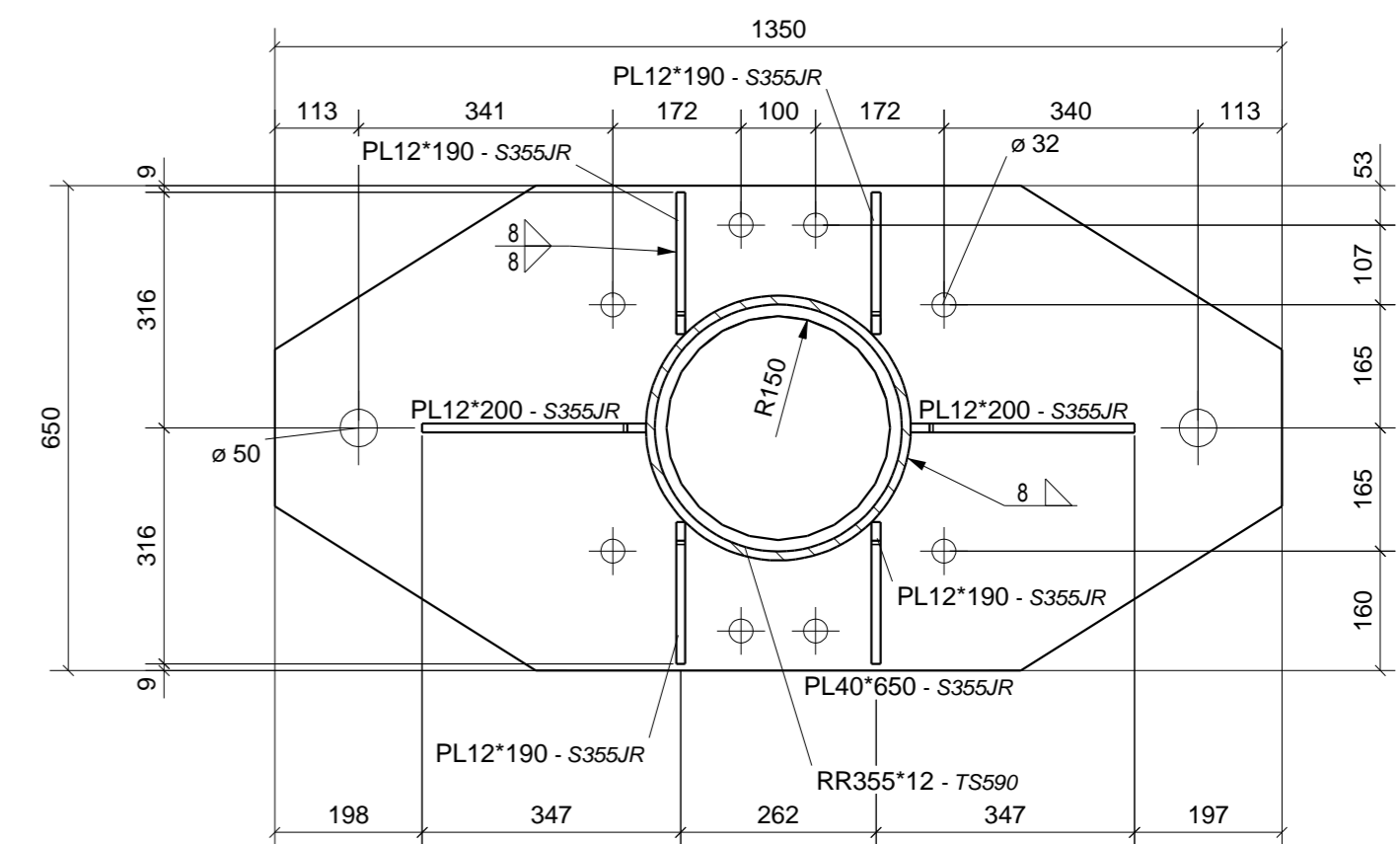
**2 SPECIMENS**



**A - A**




**B - B**



MARCA: UNITN17		Descrizione: HAUPTTRÄGER		Pezzi 2		Tratt. Sup.:	
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso	
e13	PL40*650	S355JR	1	1350	1.5	205.2	
e58	RR355*12	TS590	1	2110	2.3	214.0	
e73	PL12*190	S355JR	4	200	0.1	2.3	
e74	PL12*200	S355JR	2	300	0.1	3.6	
e107	PL25*650	S355JR	1	800	0.9	82.1	
<b>Totale :</b>					<b>5.1</b>	<b>518.0</b>	

revisione secondo indicazioni del 13-10-09 LUKAS H. 14-10-2009 A  
 ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE NAME - NOME DATUM - DATA INDEX

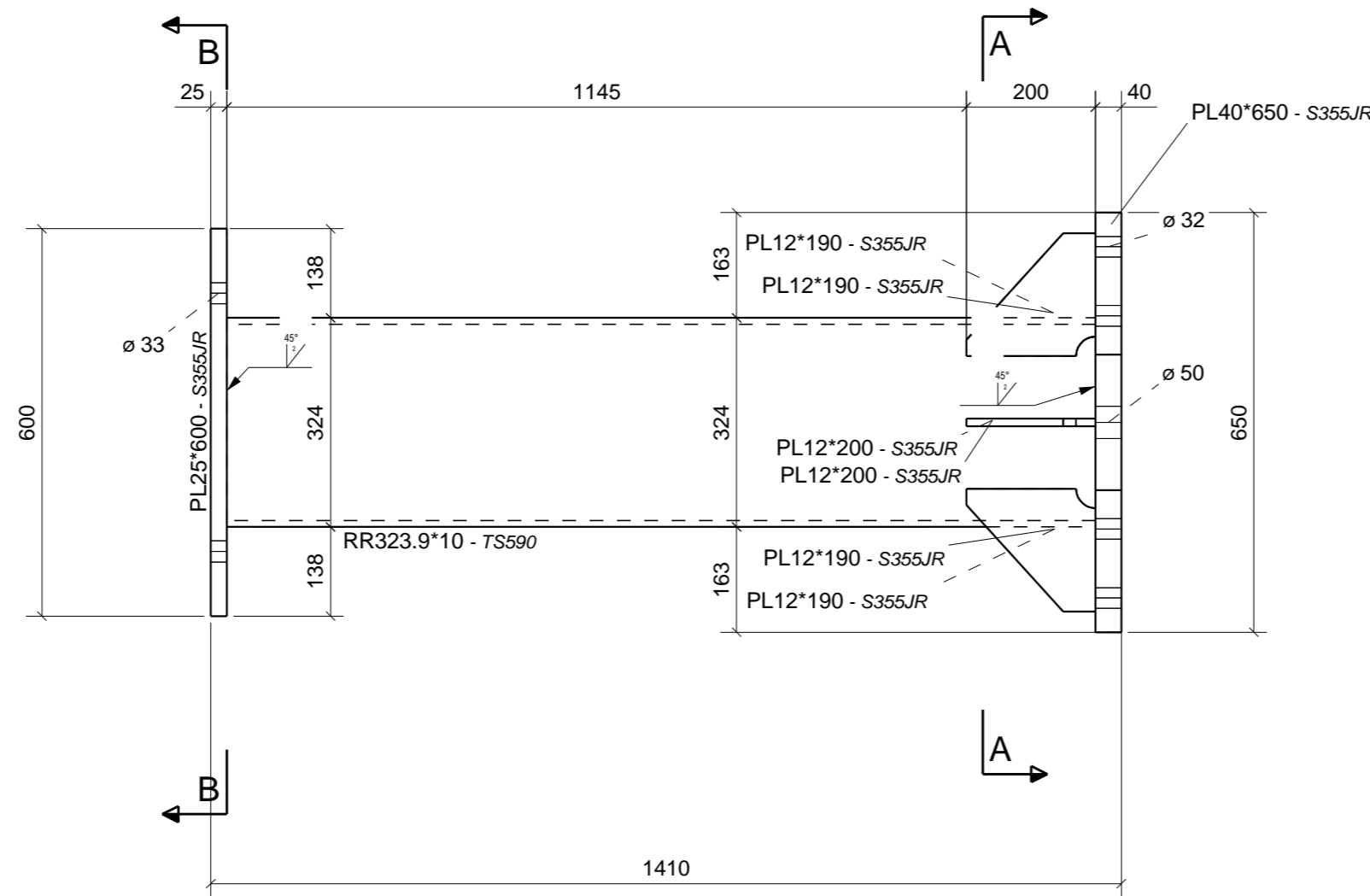
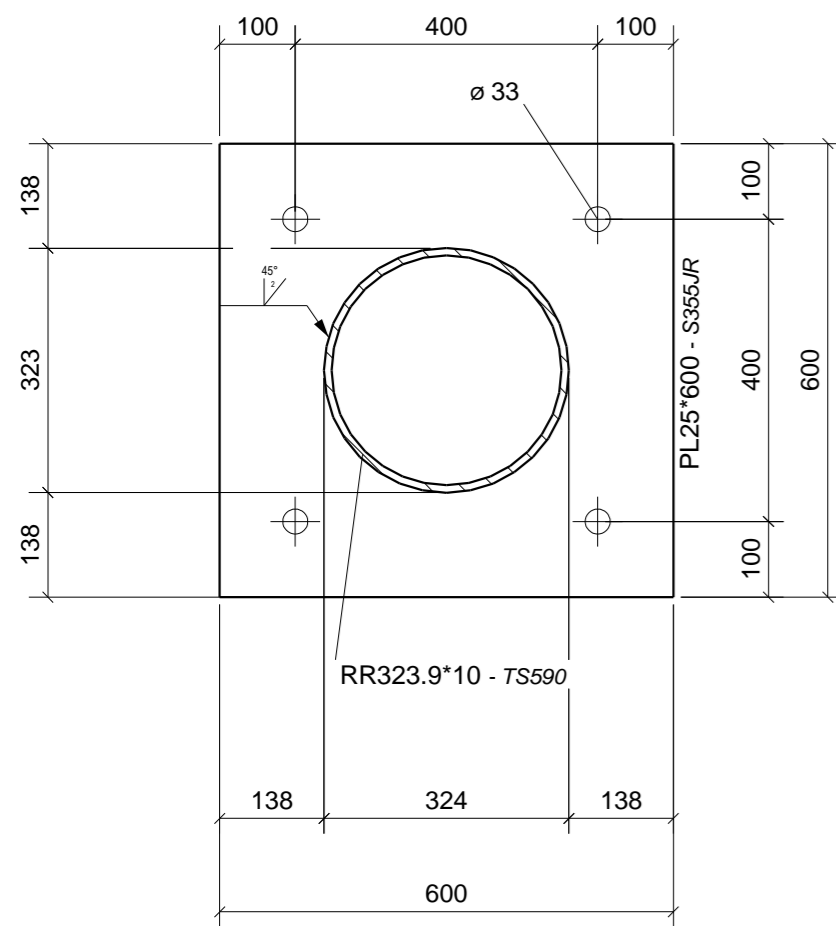
zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte : Standard Stahlbau Pichler  
 scostamenti ammessi per misure senza tolleranze / saldature :

Cliente: UNIVERSITA TRENTO		Oggetto: ATTEL		Comessa: 1559		Data: 13.11.2009		 STAHLBAU PICHLER GmbH-Srl Tel: 0471/065000 - Fax: 0471/065001 E-MAIL: info@stahlbaupichler.com 39100 BOZEN/BOLZANO		Disegno: UNITN17	
Reparto: S		Scala: 1:10		Gez./Dis.: LUKAS H.		Geprüft / Controllato: TB: Zelger				Datum/Data: ST: Sommariva	

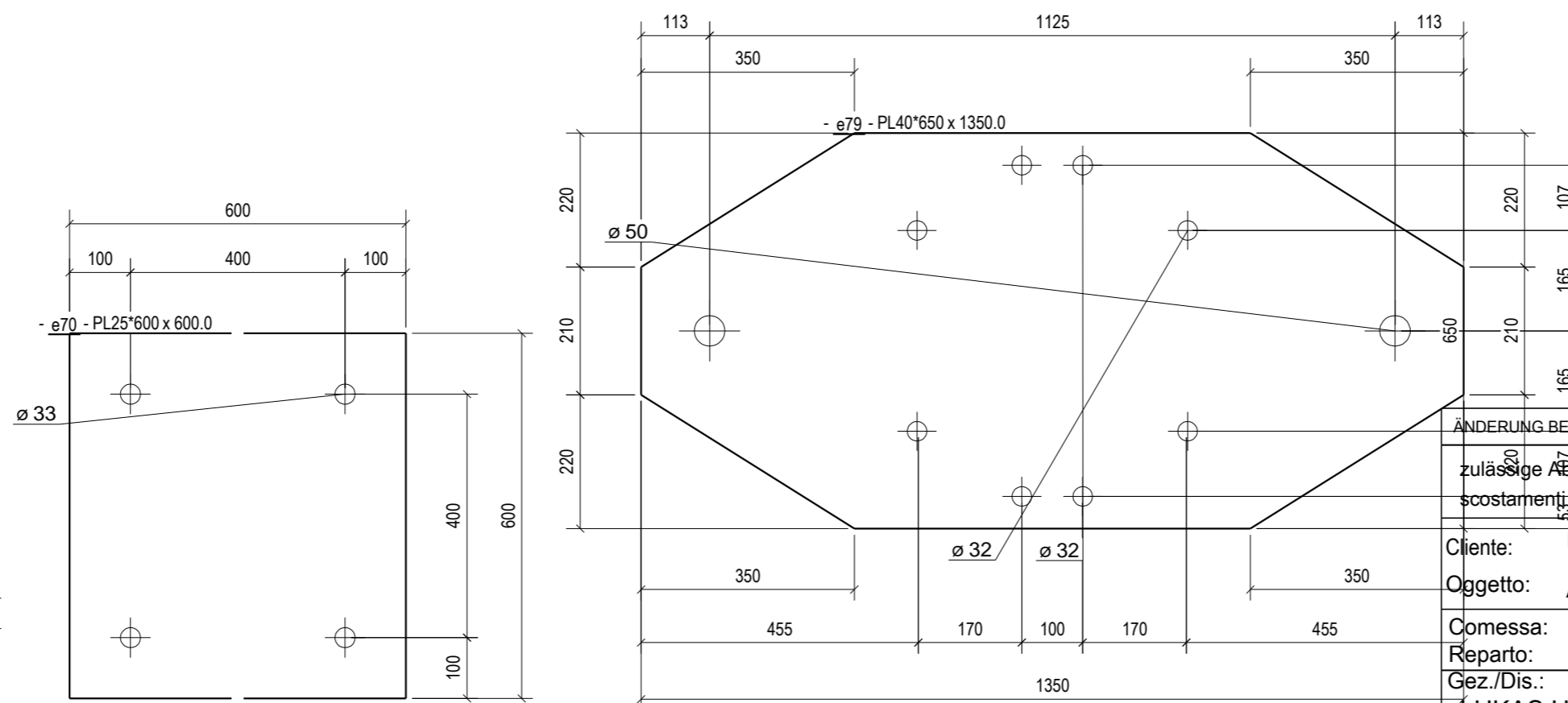
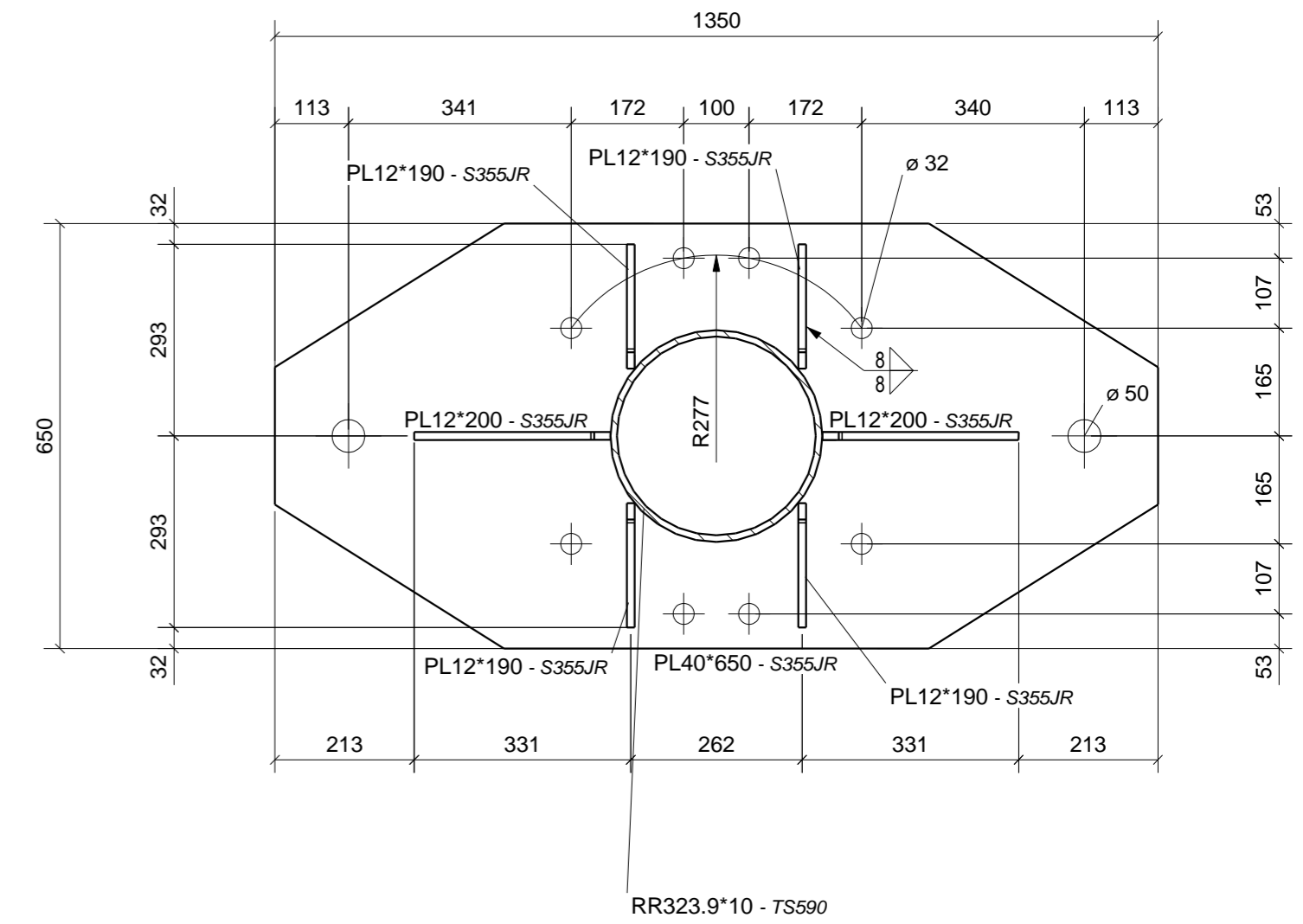
LOTTO	PEZZI
2	2



**B - B**



**A - A**



MARCA: UNITN15		Descrizione: HAUPTTRÄGER		Pezzi 2		Tratt.Sup.:	
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso	
e70	PL25*600	S355JR	1	600	0.8	70.6	
e73	PL12*190	S355JR	4	200	0.1	2.3	
e74	PL12*200	S355JR	2	300	0.1	3.6	
e79	PL40*650	S355JR	1	1350	1.6	227.2	
e91	RR323.9*10	TS590	1	1345	1.4	104.1	
<b>Totale :</b>					<b>4.1</b>	<b>418.6</b>	

ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE NAME - NOME DATUM - DATA INDEX

zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

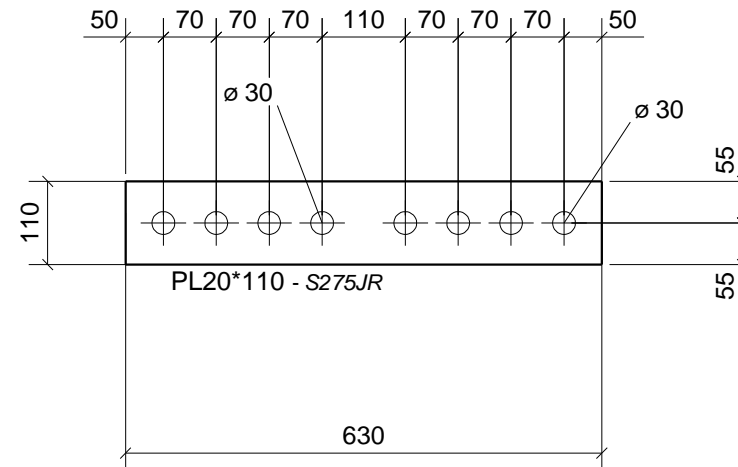
Standard Stahlbau Pichler

Ciente: **UNIVERSITA TRENTO**  
 Oggetto: **ATEL**  
 Comessa: **1559** Data: **01.09.2009**  
 Reparto: **S** Scala: **1:10**  
 Gez./Dis.: **LUKAS H.** Geprüft / Controllato: **TB: Zelger** Datum/Data: **ST:Sommariva**

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 39100 BOZEN/SOLZANO

LOTTO	PEZZI
2	2

# 24 SPECIMENS



MARCA: <b>UNITN14</b>	Descrizione: Blech	Pezzi 24	Tratt.Sup.:			
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
UNITN14	PL20*110	S275JR	1	630	0.2	10.9
Totale :					0.2	10.9

ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE      NAME - NOME      DATUM - DATA      INDEX

zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

Standard Stahlbau Pichler

Cliente: **UNIVERSITA TRENTO**

Oggetto: **ATTEL**

Comessa: **1559**

Data: **27.08.2009**

Reparto: **S**

Scala: **1:10**



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Disegno:  
**UNITN14**

Gez./Dis.:  
**LUKAS H.**

Geprüft / Controllato:  
TB: **Zelger**

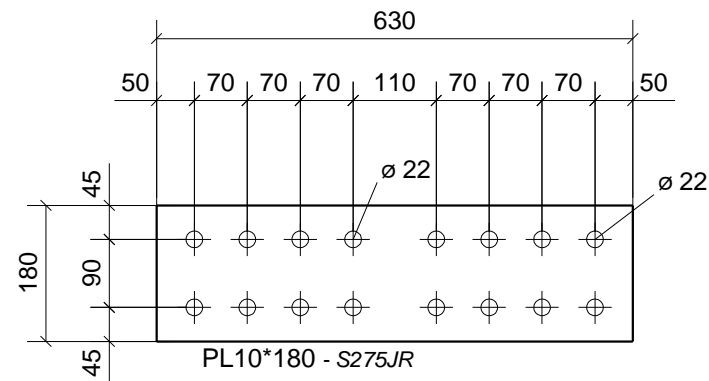
Datum/Data:

Geprüft / Controllato:  
ST: **Sommariva**

Datum/Data:

LOTTO	PEZZI
2	24

# 12 SPECIMENS



MARCA: <b>UNITN13</b>	Descrizione: Blech	Pezzi 12	Tratt.Sup.:			
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
UNITN13	PL10*180	S275JR	1	630	0.2	8.9
Totale :					0.2	8.9

ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE      NAME - NOME      DATUM - DATA      INDEX

zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

Standard Stahlbau Pichler

Cliente: **UNIVERSITA TRENTO**

Oggetto: **ATTEL**

Comessa: **1559**

Data: **27.08.2009**

Reparto: **S**

Scala: **1:10**



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Disegno:  
**UNITN13**

Gez./Dis.:  
**LUKAS H.**

Geprüft / Controllato:  
TB: **Zelger**

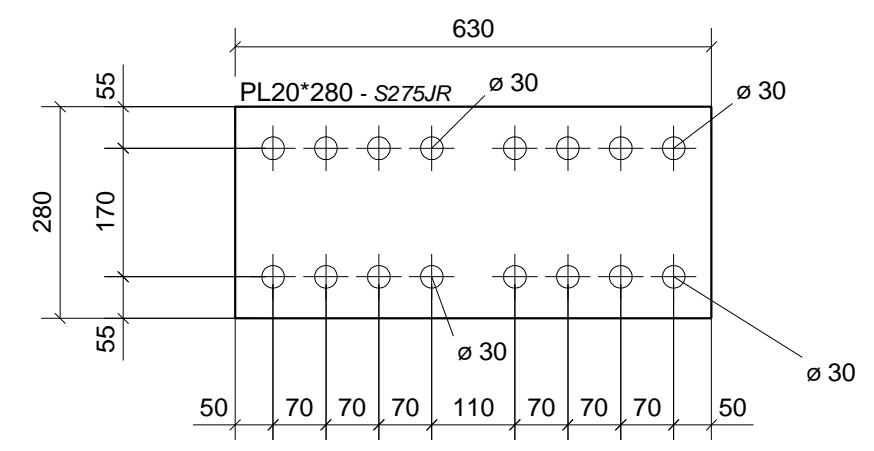
Datum/Data:

Geprüft / Controllato:  
ST: **Sommariva**



Datum/Data:

LOTTO	PEZZI
2	12

# 12 SPECIMENS

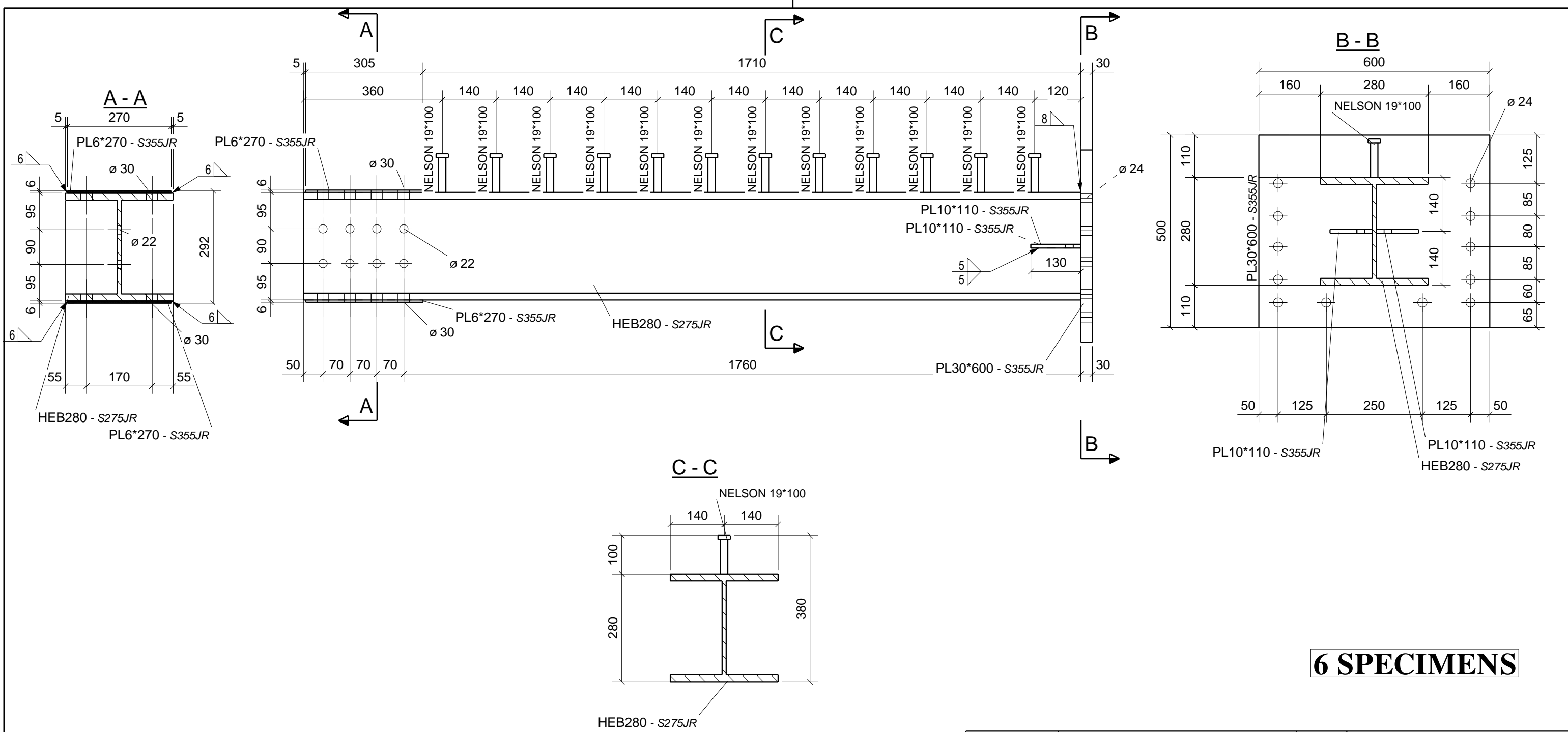


MARCA:	Descrizione:	Pezzi	Tratt.Sup.:			
UNITN12	Blech	12				
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
UNITN12	PL20*280	S275JR	1	630	0.4	27.7
Totale :					0.4	27.7

ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE		NAME - NOME		DATUM - DATA		INDEX	
zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte : scostamenti ammessi per misure senza tolleranze / saldature :						Standard Stahlbau Pichler	
Cliente:	UNIVERSITA TRENTO						
Oggetto:	ATTEL						
Comessa:	1559	Data:	01.09.2009				
Reparto:	S	Scala:	1:10				
Gez./Dis.:	LUKAS H.	Geprüft / Controllato:	Datum/Data:		Geprüft / Controllato:	Datum/Data:	
		TB: Zelger			ST: Sommariva		

LOTTO	PEZZI
2	12

Disegno:  
**UNITN12**



## 6 SPECIMENS

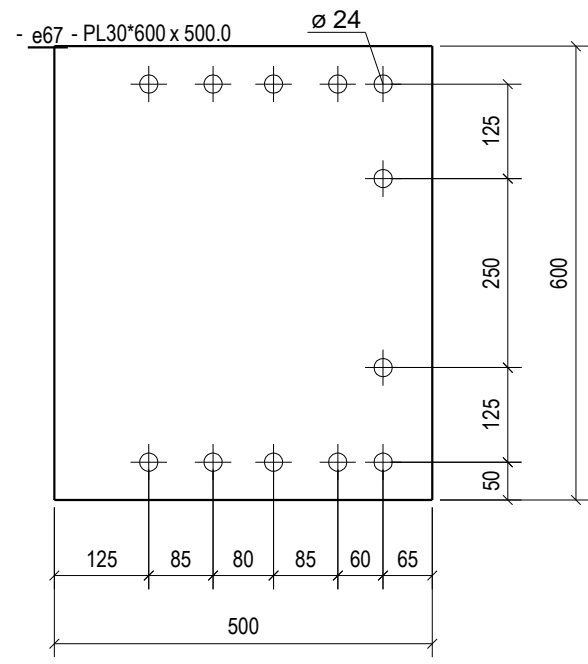
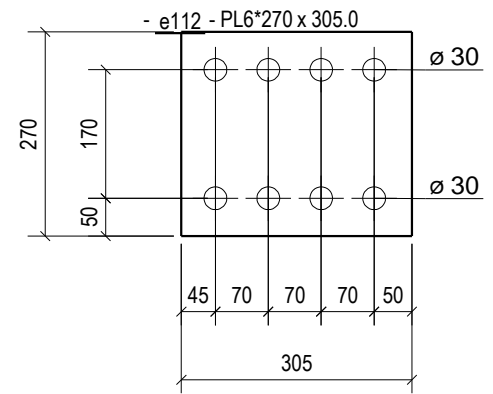
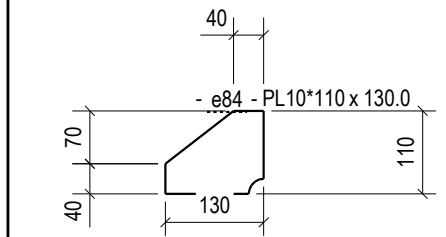
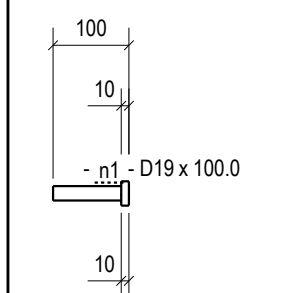
MARCA:	Descrizione:	Pezzi	Tratt.Sup.:			
UNITN11	UNILG	6				
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
e67	PL30*600	S355JR	1	500	0.7	70.6
e84	PL10*110	S355JR	2	130	0.0	0.9
e94	HEB280	S275JR	1	2020	3.3	207.7
e112	PL6*270	S355JR	2	305	0.2	3.9
n1	D19	S355JR	12	100	0.0	0.2
<b>Totale :</b>					<b>4.4</b>	<b>290.7</b>

ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE NAME - NOME DATUM - DATA INDEX

zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

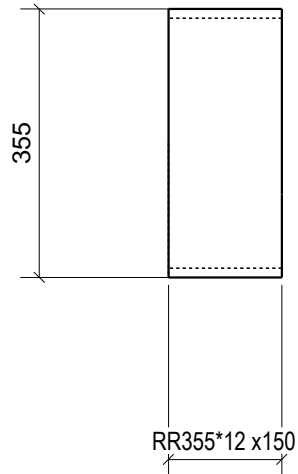
**Standard Stahlbau Pichler**

Cliente: <b>UNIVERSITA TRENTO</b> Oggetto: <b>ATTEL</b>	Comessa: <b>1559</b> Reparto: <b>S</b>	Data: <b>01.09.2009</b> Scala: <b>1:10</b>	 STAHLBAU PICHLER Tel.0471/065000 - Fax : 0471/065001 E-MAIL: info@stahlbaupichler.com 39100 BOZEN/BOLZANO	
Gez./Dis.: <b>LUKAS H.</b>	Geprüft / Controllato: <b>TB: Zelger</b>	Datum/Data:		



LOTTO	PEZZI
2	6





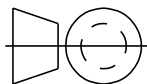
LOTTO	PEZZI
2	1

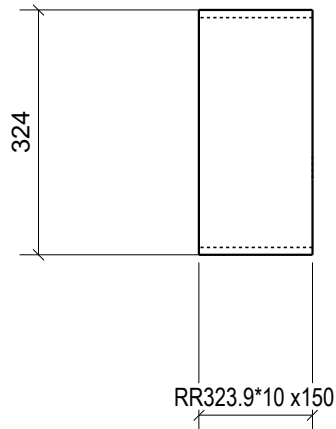
Quantità	per Marca
1	UNILG41

Cliente: UNIVERSITA TRENTO	Comessa: 1559	Reparto: S	Disegno: HAUPTTRÄGER
Descrizione: HAUPTTRÄGER	Profilo: RR355*12	Quantità: 1	Peso: 15.1
Lunghezza: 150	Posizione: UNILG41	Data: 28.09.2009	Modifica:
Superficie: 0.17			Rev:
Materiale: TS590			





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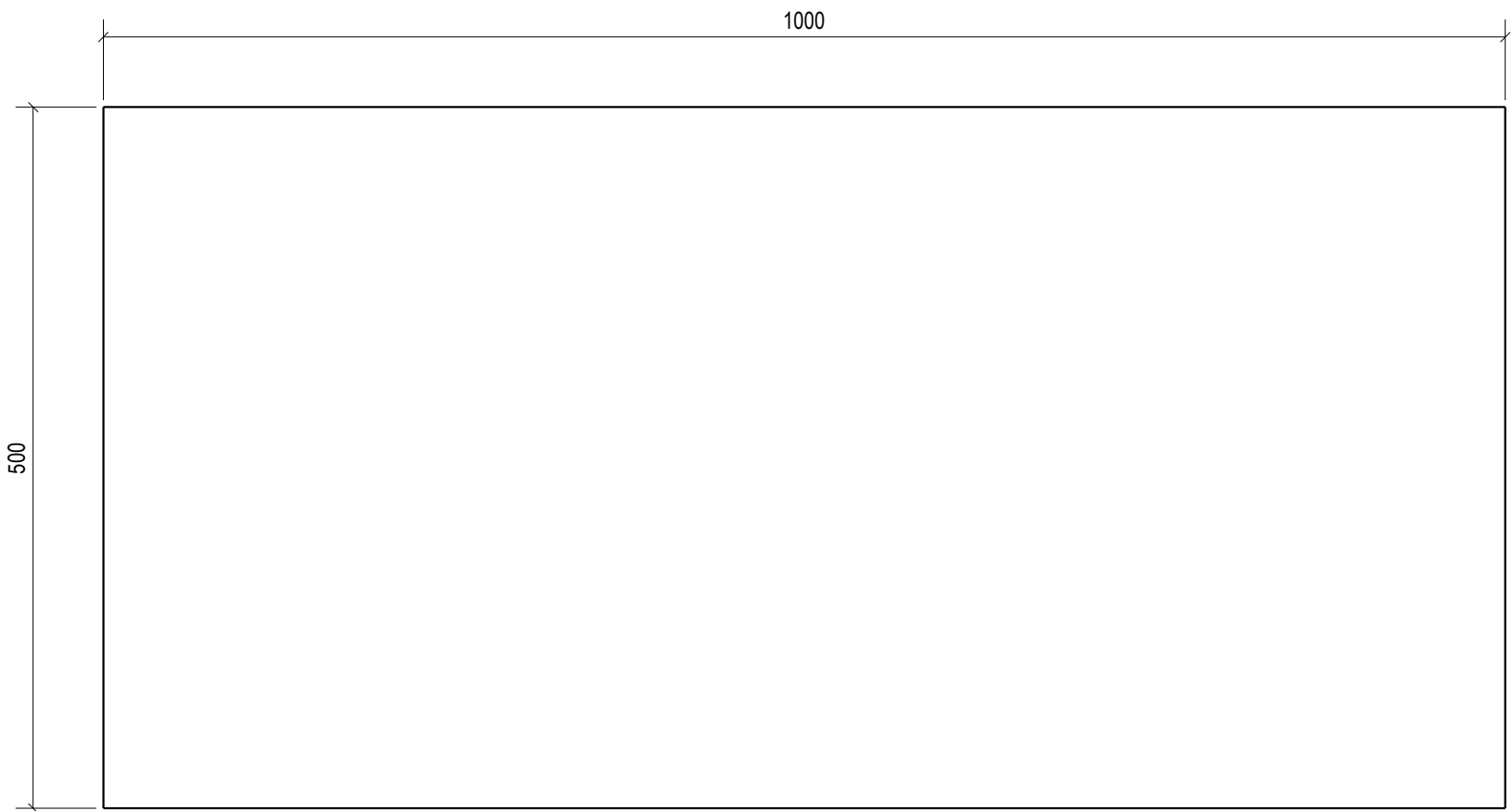




LOTTO	PEZZI
2	1

Quantità	per Marca
1	UNILG40

Cliente: UNIVERSITA TRENTO Descrizione: HAUPTTRÄGER Lunghezza: 150 Superficie: 0.16 Materiale: TS590	Comessa: 1559 Reparto: S Disegno: HAUPTTRÄGER	Profilo: <b>RR323.9*10</b> Posizione: <b>UNILG40</b>	Quantità: 1 Peso: 11.6 Data: 28.09.2009 Modifica:	Rev:	 <b>STAHLBAU PICHLER</b> GmbH-Srl <small>Tel.0471/065000 - Fax : 0471/065001 E-MAIL: info@stahlbaupichler.com 39100 BOZEN/BOLZANO</small>	
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LOTTO	PEZZI
2	1

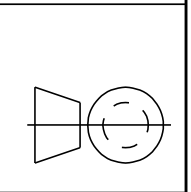
Quantità	per Marca
1	UNILG39

Cliente: UNIVERSITA TRENTO  
 Descrizione: PLATTE  
 Lunghezza: 1000  
 Superficie: 1.08  
 Materiale: S355JR

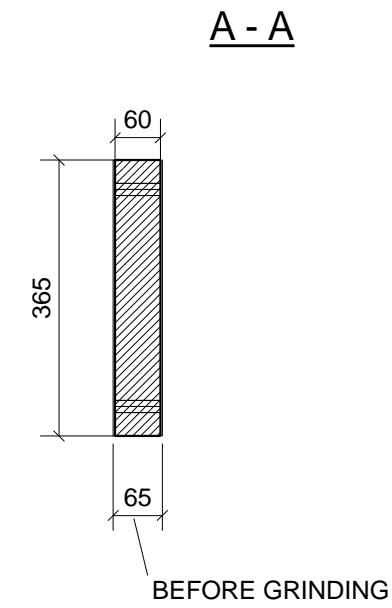
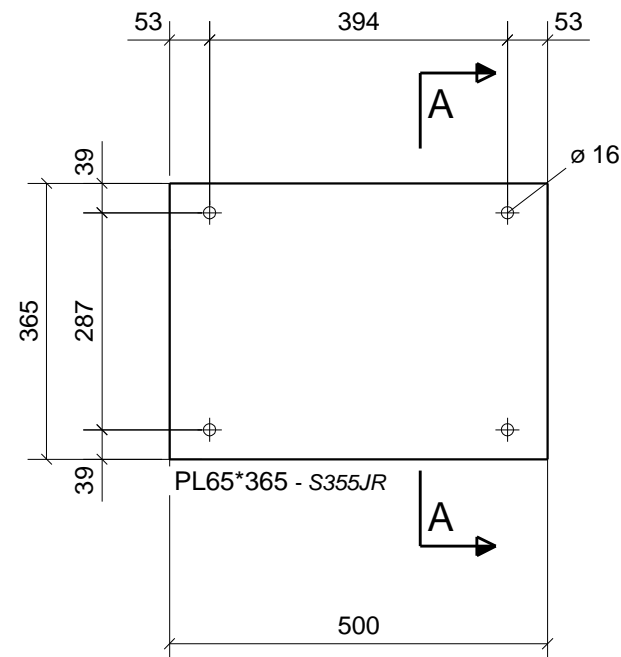
Comessa: 1559    Reparto: S  
 Profilo: **PL25\*500**  
 Posizione: **UNILG39**

Disegno: PLATTE  
 Quantità: 1  
 Peso: 98.1  
 Data: 28.09.2009  
 Modifica:                      Rev:


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# 2 SPECIMENS



MARCA: <b>UNILG38</b>	Descrizione: PLATTE	Pezzi 2	Tratt.Sup.:			
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
UNILG38	PL65*365	S355JR	1	500	0.5	86.0
Totale :					0.5	86.0

ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE NAME - NOME DATUM - DATA INDEX

zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

Standard Stahlbau Pichler

Cliente: **UNIVERSITA TRENTO**

Oggetto: **ATTEL**

Comessa: 1559

Data: 01.09.2009

Reparto: S

Scala: 1:10

**STAHLBAU PICHLER** GmbH-Srl  
Tel.0471/065000 - Fax : 0471/065001  
E-MAIL: info@stahlbaupichler.com  
39100 BOZEN/BOLZANO

Disegno:  
**UNILG38**

Gez./Dis.:  
LUKAS H.

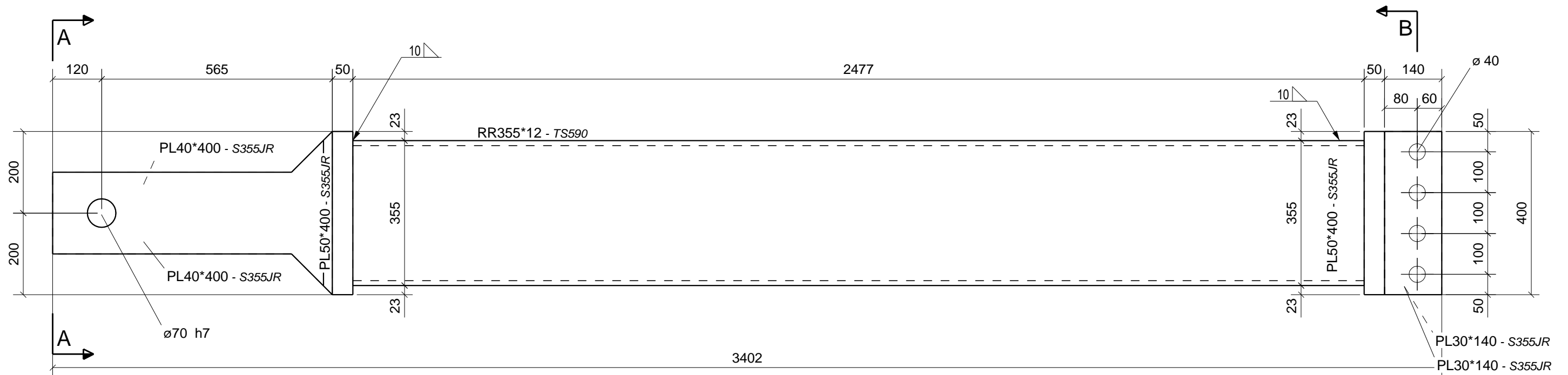
Geprüft / Controllato:  
TB: Zelger

Datum/Data:

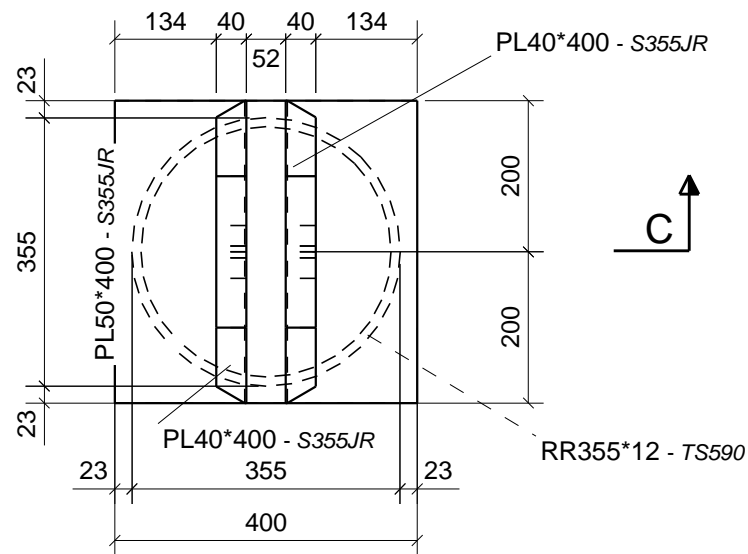
Geprüft / Controllato:  
ST: Sommariva

Datum/Data:

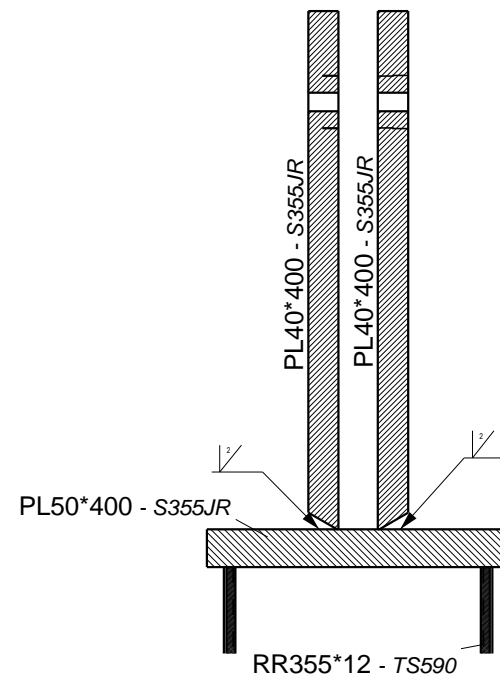
LOTTO	PEZZI
2	2



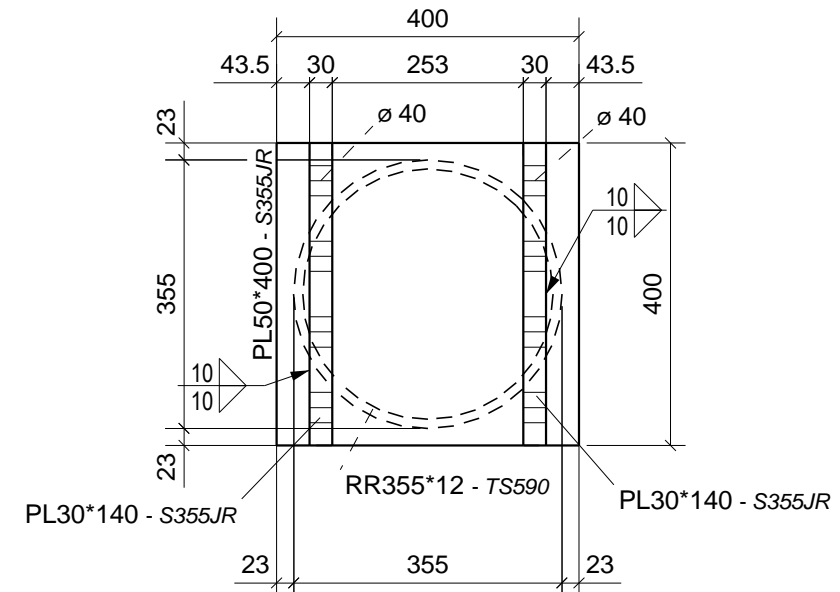
A - A



C - C

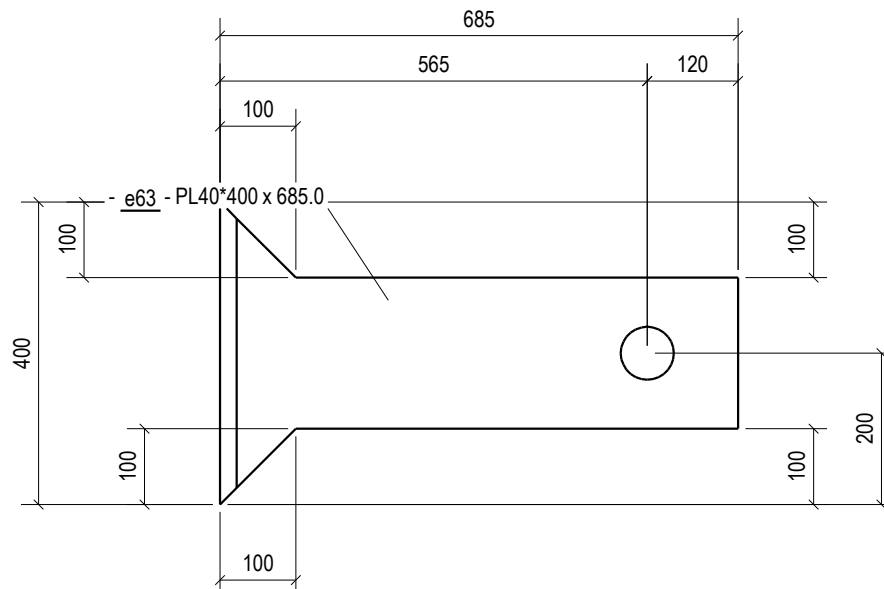
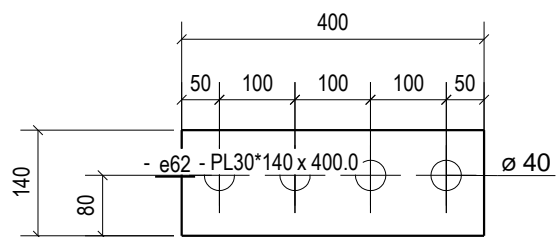


B - B



**1 SPECIMEN**

MARCA: UNILG37		Descrizione: HAUPTTRÄGER		Pezzi 1		Tratt.Sup.:	
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso	
e56	RR355*12	TS590	1	2477	2.7	249.8	
e62	PL30*140	S355JR	2	400	0.1	13.2	
e63	PL40*400	S355JR	2	685	0.4	43.7	
e108	PL50*400	S355JR	2	400	0.4	62.8	
Totale :					4.5	489.2	



LOTTO	PEZZI
2	1

revisione secondo indicazioni

ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE

zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

Standard Stahlbau Pichler

Ciente: UNIVERSITA TRENTO  
Oggetto: ATTEL

Comessa: 1559  
Reparto: S

Data: 14.10.2009  
Scala: 1:10

Gez./Dis.: LUKAS H.  
Geprüft / Controllato: TB: Zelger

Datum/Data: 28-09-2009

Geprüft / Controllato: ST: Sommariva

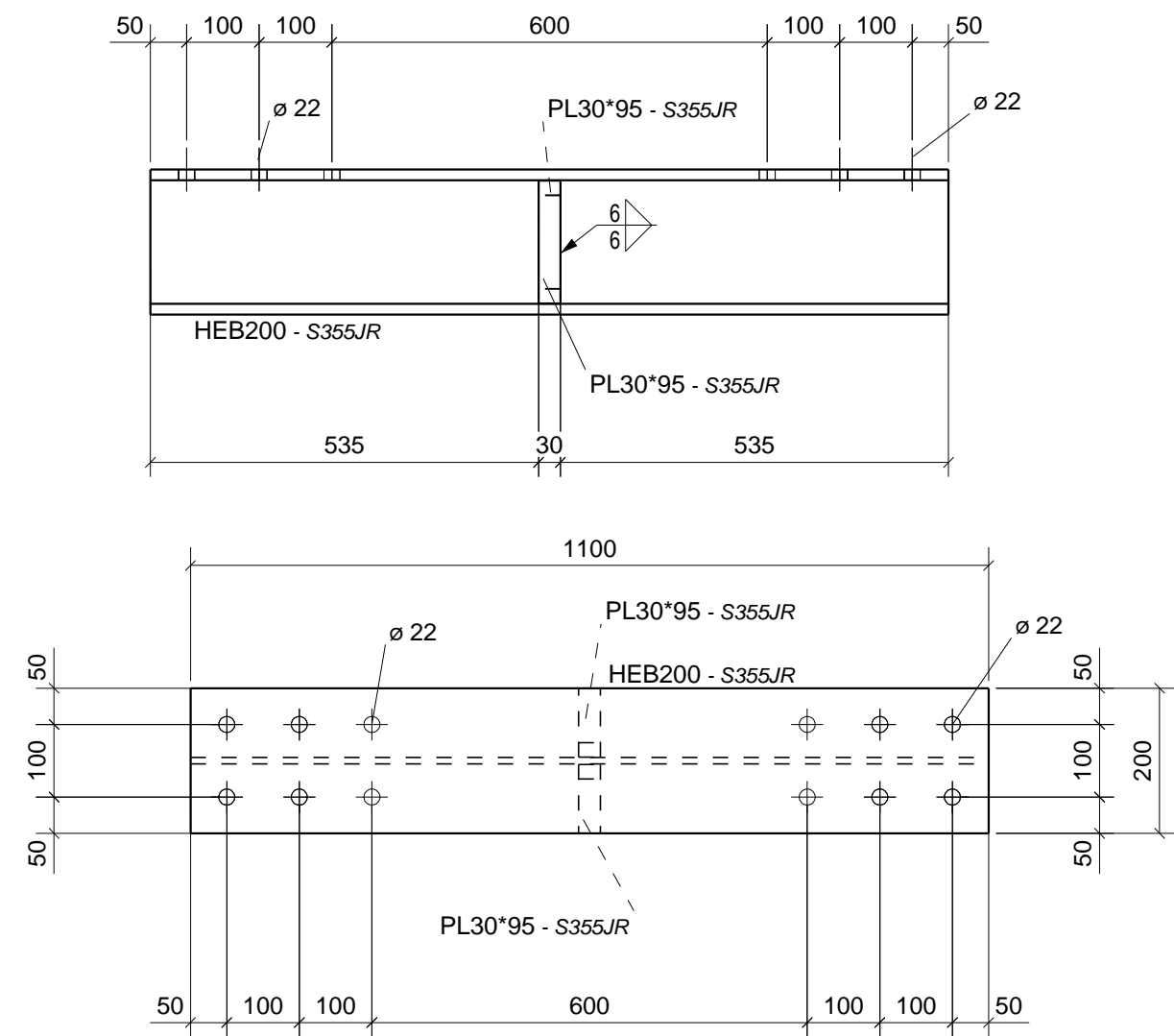
LUKAS H. 28-09-2009 A

NAME - NOME DATUM - DATA INDEX

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Tel.0471/065000 - Fax : 0471/065001  
E-MAIL: info@stahlbaupichler.com  
39100 BOZEN/BOLZANO

Disegno: UNILG37

# 2 SPECIMENS

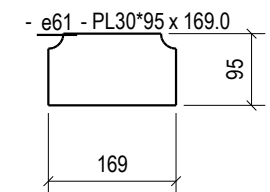


MARCA: <b>UNILG36</b>	Descrizione: <b>HAUPTTRÄGER</b>	Pezzi <b>2</b>	Tratt.Sup.:			
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
e61	PL30*95	S355JR	2	169	0.0	3.6
e101	HEB200	S355JR	1	1100	1.3	67.4
<b>Totale :</b>					<b>1.4</b>	<b>74.7</b>

ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE      NAME - NOME      DATUM - DATA      INDEX

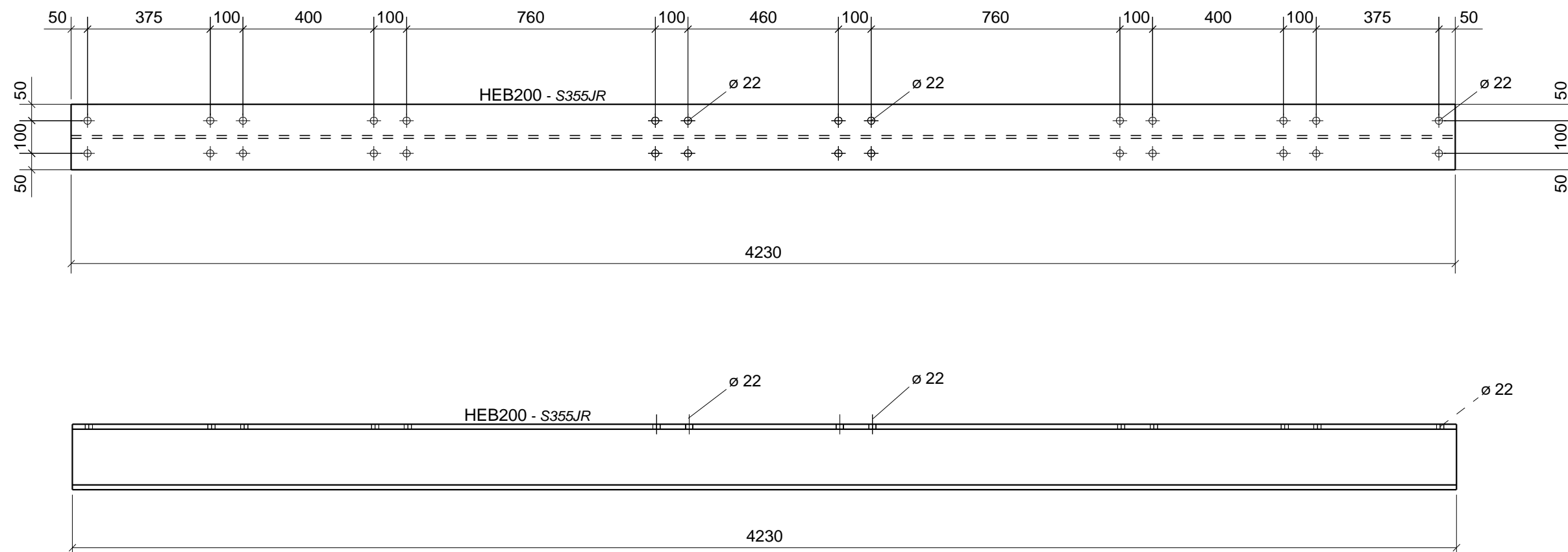
zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :      **Standard Stahlbau Pichler**

Cliente: <b>UNIVERSITA TRENTO</b> Oggetto: <b>ATTEL</b>	Comessa: <b>1559</b> Reparto: <b>S</b>	Data: <b>01.09.2009</b> Scala: <b>1:10</b>		
Gez./Dis.: <b>LUKAS H.</b>	Geprüft / Controllato: <b>TB: Zelger</b>	Datum/Data:		



LOTTO	PEZZI
2	2

# 2 SPECIMENS


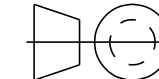



MARCA: <b>UNILG35</b>	Descrizione: <b>HAUPTTRÄGER</b>	Pezzi <b>2</b>	Tratt.Sup.:			
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
UNILG35	HEB200	S355JR	1	4230	4.9	259.3
Totale :					4.9	259.3

ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE      NAME - NOME      DATUM - DATA      INDEX

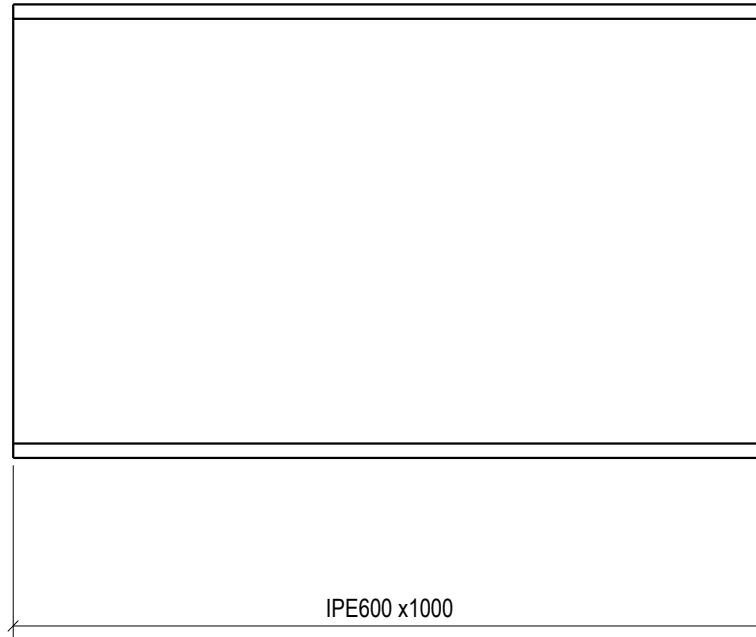
zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

Standard Stahlbau Pichler

Cliente: <b>UNIVERSITA TRENTO</b>		
Oggetto: <b>ATTEL</b>		
Comessa: <b>1559</b>	Data: <b>27.08.2009</b>	
Reparto: <b>S</b>	Scala: <b>1:15</b>	
Gez./Dis.: <b>LUKAS H.</b>	Geprüft / Controllato: <b>TB: Zelger</b>	Datum/Data:
	Geprüft / Controllato: <b>ST: Sommariva</b>	Datum/Data:

LOTTO	PEZZI
2	2

# 1 SPECIMEN



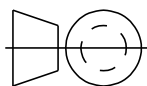
LOTTO	PEZZI
2	1

Quantità	per Marca
1	UNILG34

Cliente: UNIVERSITA TRENTO	Comessa: 1559	Reparto: S	Disegno: UNILG
Descrizione: UNILG	Profilo: IPE600	Quantità: 1	Peso: 122.5
Lunghezza: 1000	Posizione: UNILG34	Data: 12.11.2009	Modifica:
Superficie: 2.09			Rev:
Materiale: S355JR			

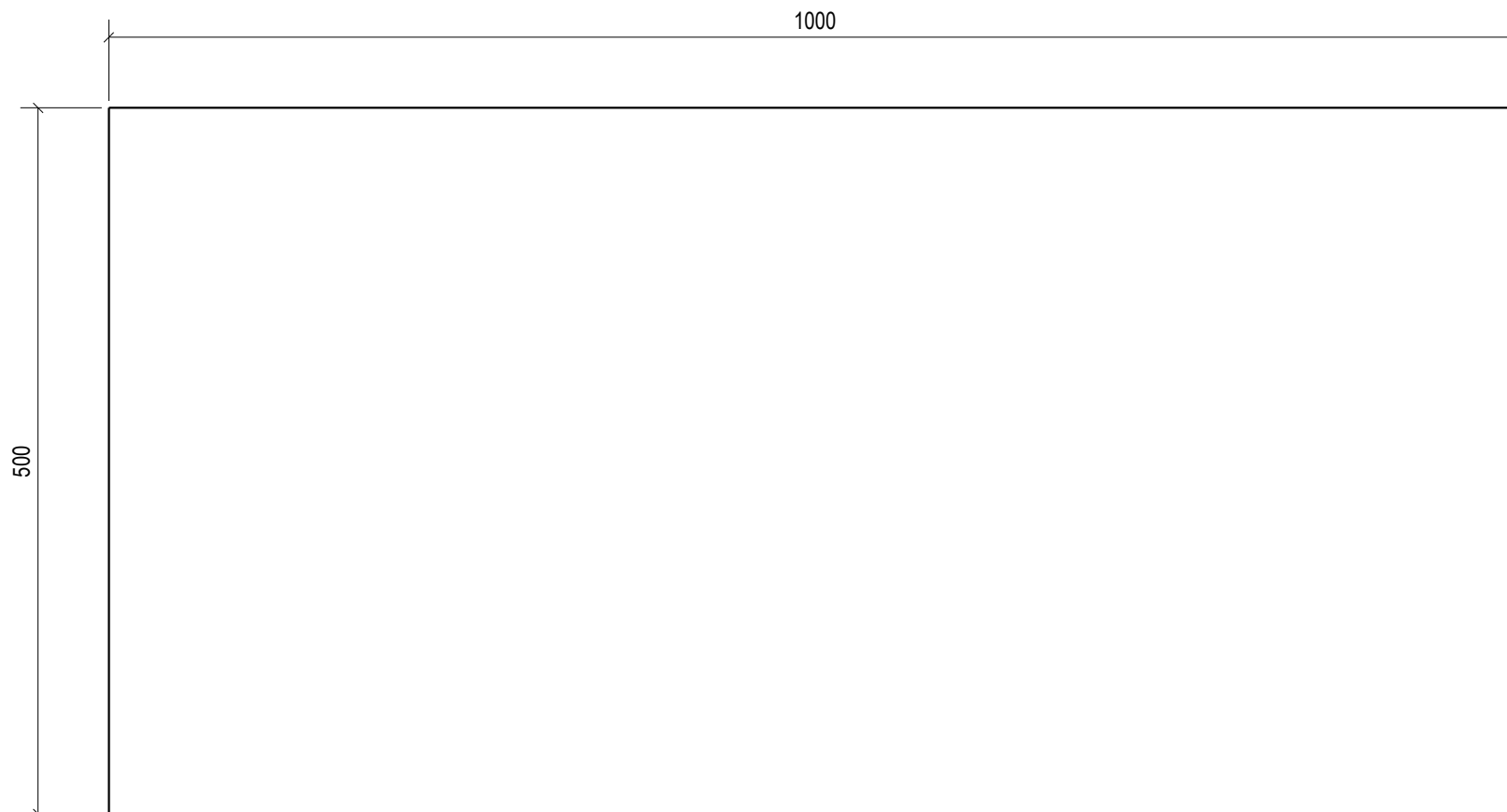


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# 1 SPECIMEN



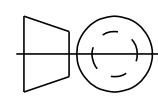
LOTTO	PEZZI
2	1

Quantità	per Marca
1	UNILG32

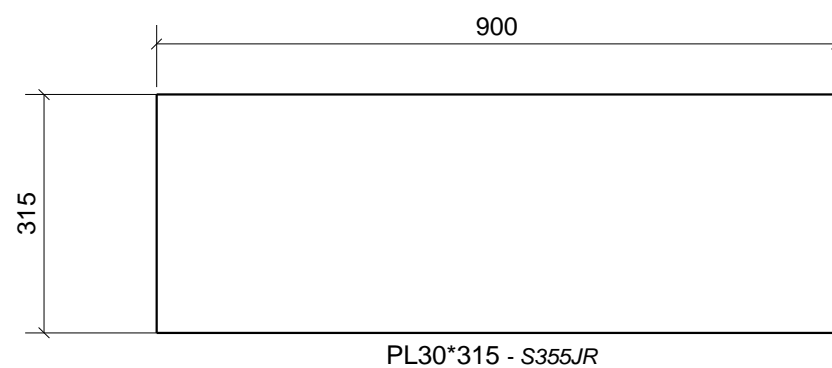
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Descrizione: PLATTE	Profilo: PL16*500	Quantità: 1	Peso: 62.8
Lunghezza: 1000	Posizione: UNILG32	Data: 28.09.2009	Modifica:
Superficie: 1.05			Rev:
Materiale: S355JR			



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# 1 SPECIMEN



MARCA: <b>UNILG31</b>	Descrizione: PLATTE	Pezzi 1	Tratt.Sup.:			
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
UNILG31	PL30*315	S355JR	1	900	0.6	66.8
Totale :					0.6	66.8

ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE      NAME - NOME      DATUM - DATA      INDEX

zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

Standard Stahlbau Pichler

Cliente: **UNIVERSITA TRENTO**

Oggetto: **ATTEL**

Comessa: **1559**

Data: **27.08.2009**

Reparto: **S**

Scala: **1:10**

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Tel. 0471/065000 - Fax : 0471/065001  
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39100 BOZEN/BOLZANO

Disegno:  
**UNILG31**

Gez./Dis.:  
**LUKAS H.**

Geprüft / Controllato:  
**TB: Zelger**

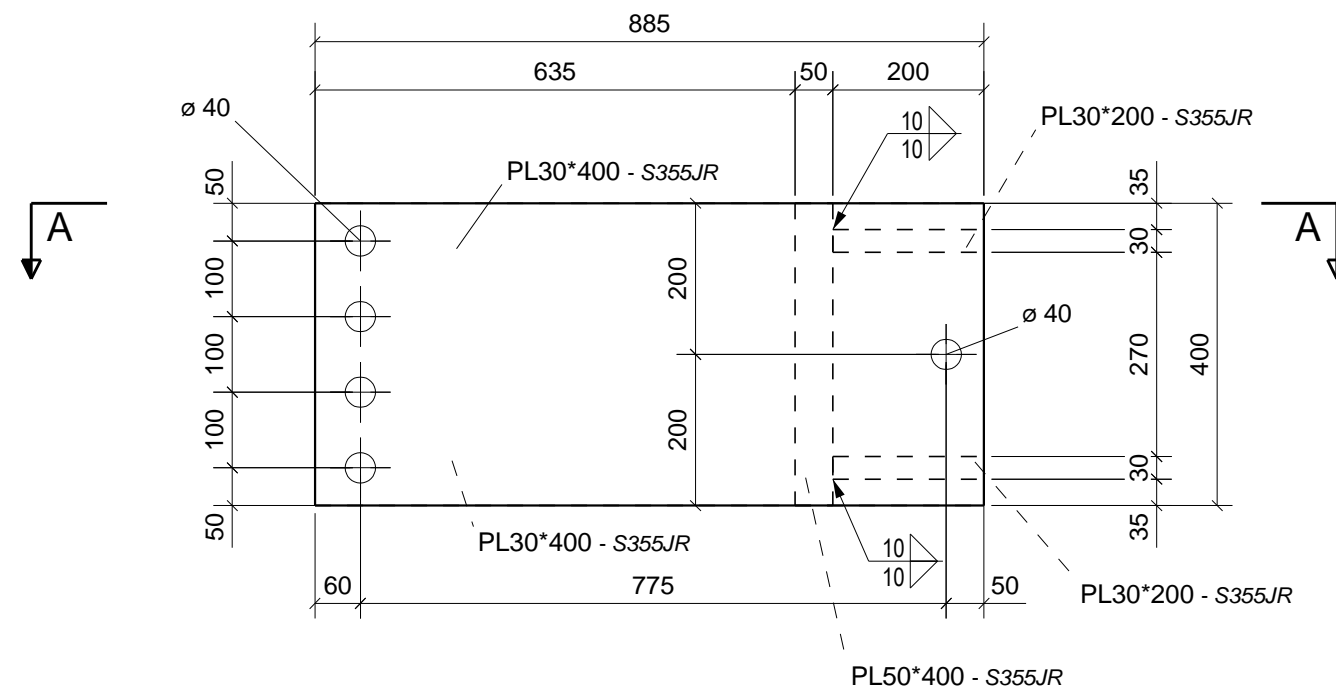
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Geprüft / Controllato:  
**ST: Sommariva**

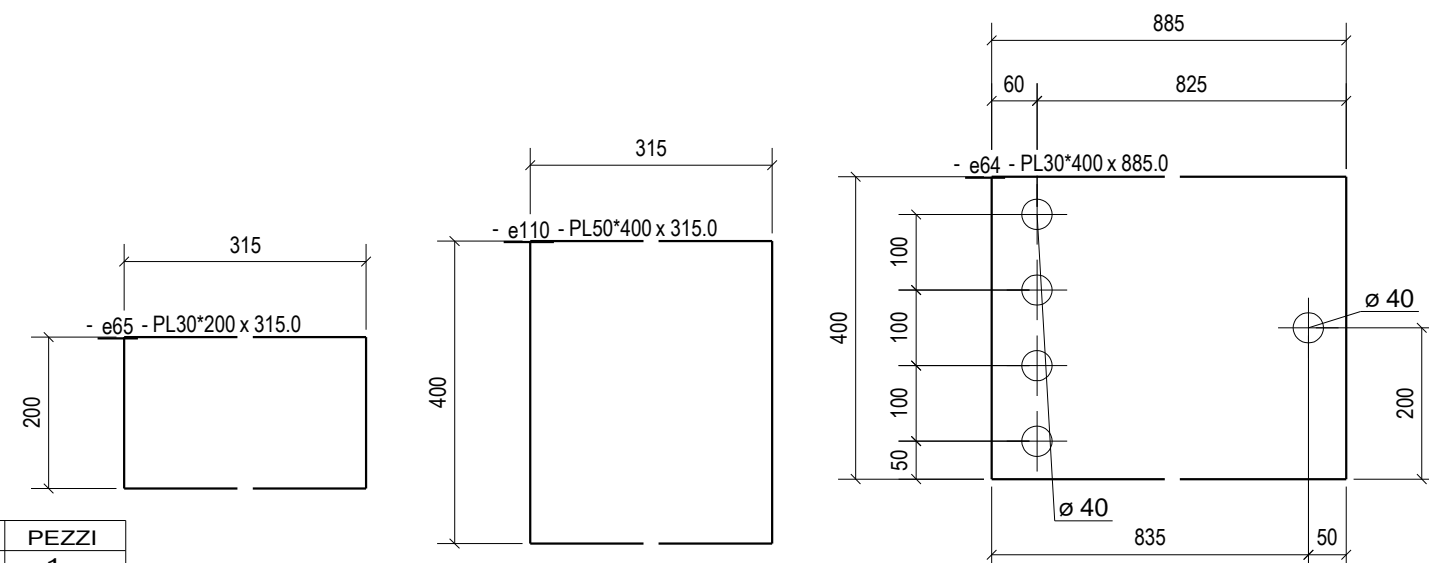
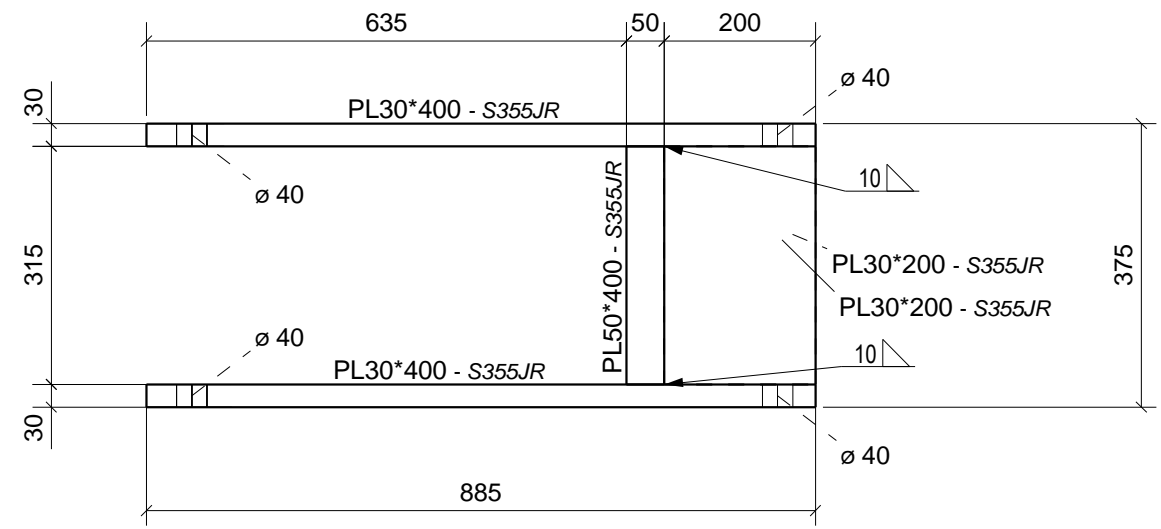
Datum/Data:

LOTTO	PEZZI
2	1


**1 SPECIMEN**



**A - A**

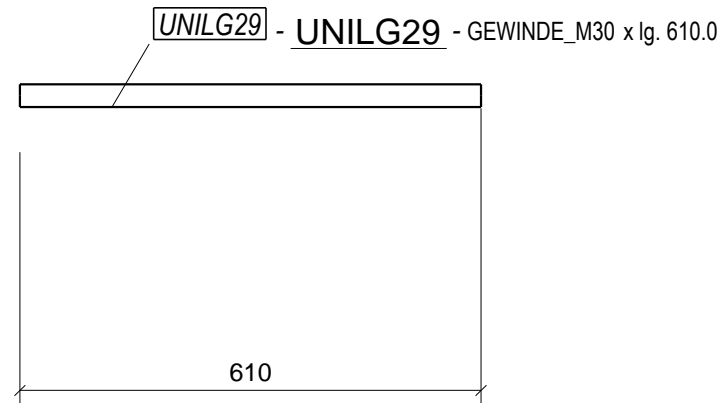


MARCA: <b>UNILG30</b>	Descrizione: <b>PLATTE</b>	Pezzi <b>1</b>	Tratt.Sup.:			
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
e64	PL30*400	S355JR	2	885	0.8	83.4
e65	PL30*200	S355JR	2	315	0.2	14.8
e110	PL50*400	S355JR	1	315	0.3	49.5
Totale :					2.2	245.9

revisione secondo indicazioni		LUKAS H.		28-09-2009		A	
ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE		NAME - NOME		DATUM - DATA		INDEX	
zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte : scostamenti ammessi per misure senza tolleranze / saldature :				<b>Standard Stahlbau Pichler</b>			
Cliente: <b>UNIVERSITA TRENTO</b>		Oggetto: <b>ATTEL</b>				 Disegno: <b>UNILG30</b>	
Comessa: 1559		Data: 14.10.2009					
Reparto: S		Scala: 1:10		STAHLBAU PICHLER <small>Tel.0471/065000 - Fax : 0471/065001 E-MAIL: info@stahlbaupichler.com 39100 BOZEN/BOLZANO</small>			
Gez./Dis.: LUKAS H.		Geprüft / Controllato: TB: Zelger		Datum/Data:		Geprüft / Controllato: ST: Sommariva	

LOTTO	PEZZI
2	1

# 4 SPECIMENS


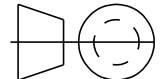


MARCA: <b>UNILG29</b>	Descrizione: <b>GEWINDESTANGE</b>	Pezzi <b>4</b>	Tratt.Sup.:			
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
UNILG29	GEWINDE_M30	10.9	1	610	0.1	3.2
Totale :					0.1	3.2

revisione secondo indicazioni			LUKAS H.	28-09-2009	A
ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE			NAME - NOME	DATUM - DATA	INDEX

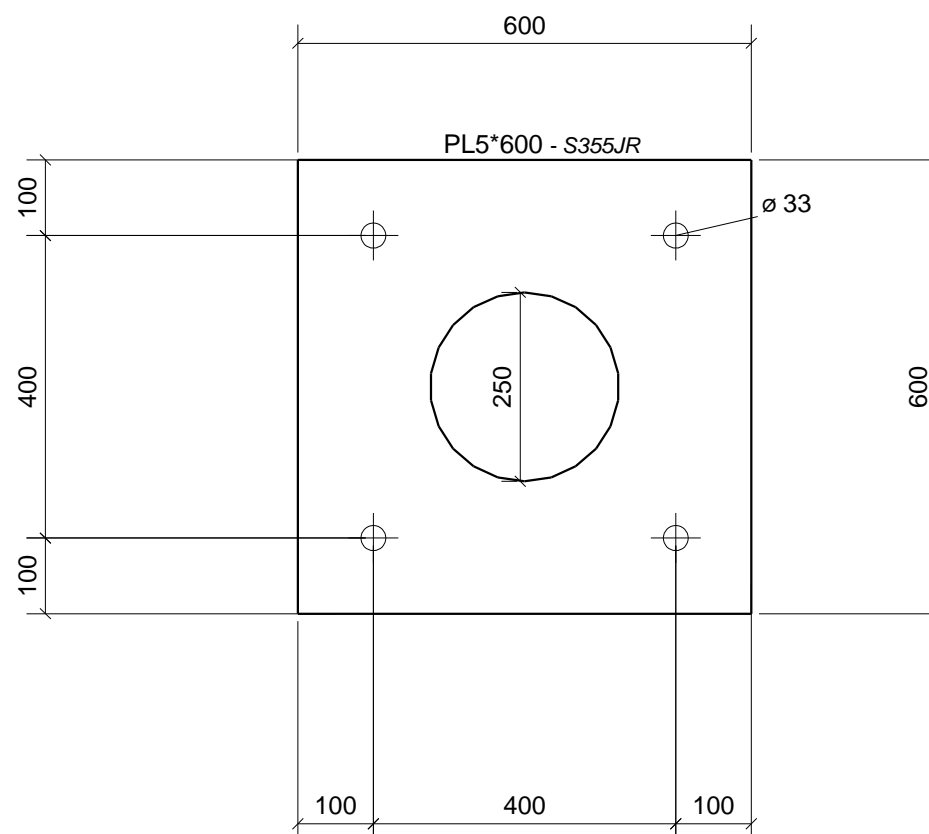
zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

**Standard Stahlbau Pichler**

Cliente: <b>UNIVERSITA TRENTO</b>	 <p>STAHLBAU PICHLER GmbH-Srl Tel.0471/065000 - Fax : 0471/065001 E-MAIL: info@stahlbaupichler.com 39100 BOZEN/BOLZANO</p>	 <p>Disegno: <b>UNILG29</b></p>		
Oggetto: <b>ATTEL</b>				
Comessa: <b>1559</b>	Data: <b>24.09.2009</b>			
Reparto:	Scala: <b>1:10</b>			
Gez./Dis.:	Geprüft / Controllato: TB: Zelger	Datum/Data:	Geprüft / Controllato: ST: Sommariva	Datum/Data:

LOTTO	PEZZI
2	4

# 1 SPECIMEN



MARCA: <b>UNILG28</b>	Descrizione: PLATTE	Pezzi 1	Tratt.Sup.:			
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
UNILG28	PL5*600	S355JR	1	600	0.6	12.2
Totale :					0.6	12.2

ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE      NAME - NOME      DATUM - DATA      INDEX

zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

Standard Stahlbau Pichler

Cliente: **UNIVERSITA TRENTO**

Oggetto: **ATTEL**

Comessa: **1559**

Data: **01.09.2009**

Reparto: **S**

Scala: **1:10**

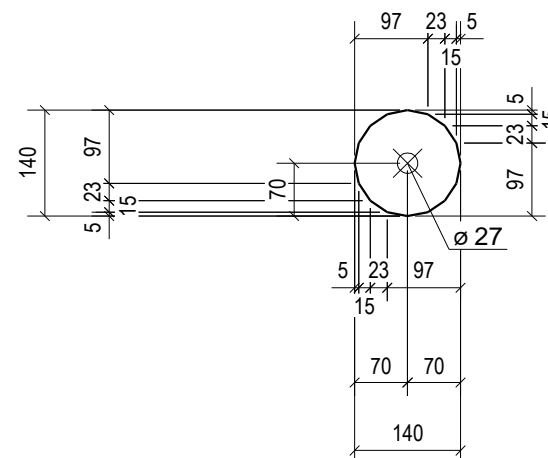
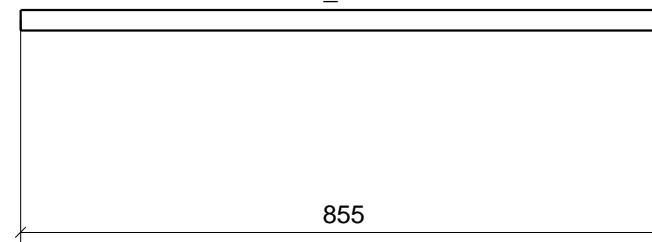
Disegno:  
**UNILG28**

LOTTO	PEZZI
2	1

Gez./Dis.: <b>LUKAS H.</b>	Geprüft / Controllato: TB: Zelger	Datum/Data:	Geprüft / Controllato: ST: Sommariva	Datum/Data:
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# 8 SPECIMENS

GEWINDE\_M27 - 10.9



MARCA: <b>UNILG13</b>	Descrizione: PLATTE	Pezzi 16	Tratt.Sup.:			
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
MARCA: <b>UNILG27</b>	Descrizione: GEWINDESTANGE	Pezzi 8	Tratt.Sup.:			
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
UNILG13	PL25*140	S355JR	1	140	0.0	2.9
UNILG27	GEWINDE_M27	10.9	1	855	0.1	3.7
Totale :					0.1	6.6

revisione secondo indicazioni		LUKAS H.	28-09-2009	A
ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE		NAME - NOME	DATUM - DATA	INDEX

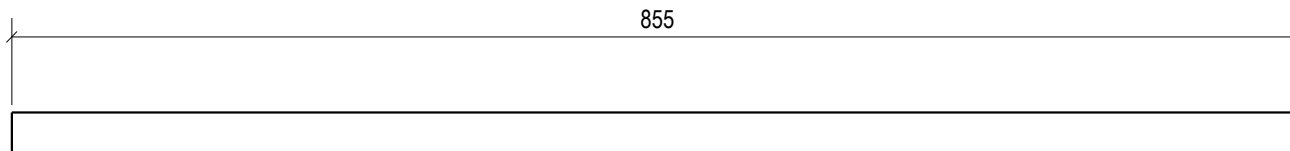
zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

Standard Stahlbau Pichler

Cliente:	UNIVERSITA TRENTO		 STAHLBAU <b>PICHLER</b> GmbH-Srl <small>STRADE 10, 39100 BOZEN/BOLZANO</small> Tel.0471/065000 - Fax : 0471/065001 E-MAIL: info@stahlbaupichler.com	
Oggetto:	ATTEL			
Comessa:	1559	Data:	24.09.2009	
Reparto:	S	Scala:	1:10	
Gez./Dis.:	LUKAS H.	Geprüft / Controllato:	TB: Zelger	Datum/Data:
		Geprüft / Controllato:	ST: Sommariva	Datum/Data:

LOTTO	PEZZI
2	24

# 8 SPECIMENS



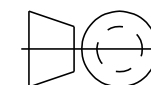
LOTTO	PEZZI
2	8

Quantità	per Marca
8	UNILG27

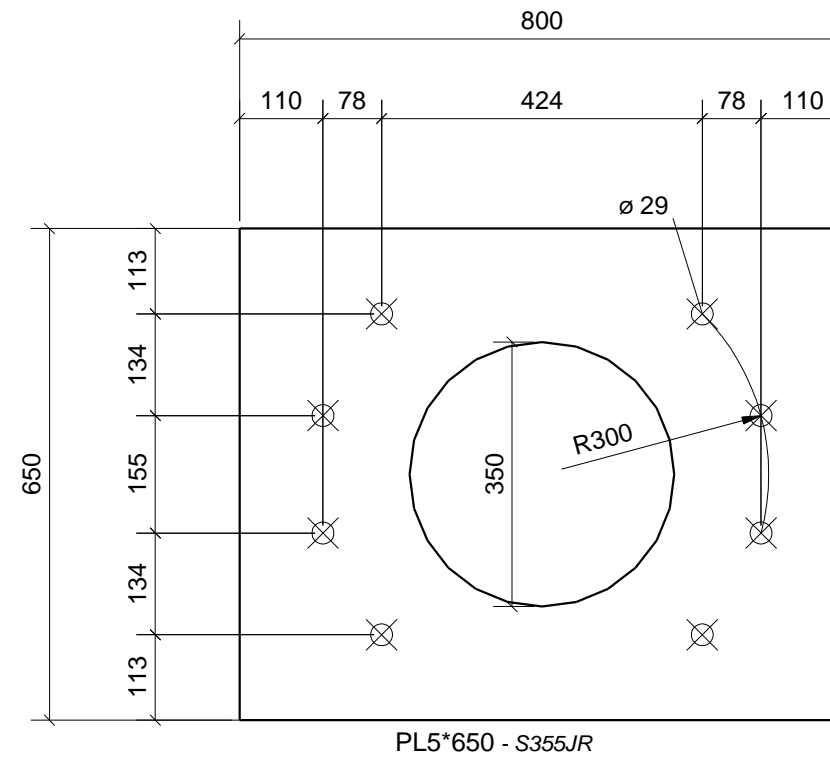
Cliente: UNIVERSITA TRENTO	Comessa: 1559	Reparto: S	Disegno: GEWINDESTAN
Descrizione: GEWINDESTANGE	Profilo: GEWINDE M27		Quantità: 8
Lunghezza: 855	Posizione: UNILG27		Peso: 3.8
Superficie: 0.07			Data: 13.11.2009
Materiale: 10.9			Modifica: 28-09-2009 Rev: A



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# 1 SPECIMEN



MARCA: <b>UNILG26</b>	Descrizione: PLATTE	Pezzi 1	Tratt.Sup.:			
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
UNILG26	PL5*650	S355JR	1	800	0.9	16.7
Totale :					0.9	16.7

ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE      NAME - NOME      DATUM - DATA      INDEX

zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

Standard Stahlbau Pichler

Cliente: **UNIVERSITA TRENTO**

Oggetto: **ATTEL**

Comessa: 1559

Data: 27.08.2009

Reparto: S

Scala: 1:10

Disegno:  
**UNILG26**

Gez./Dis.:  
LUKAS H.

Geprüft / Controllato:  
TB: Zelger

Datum/Data:

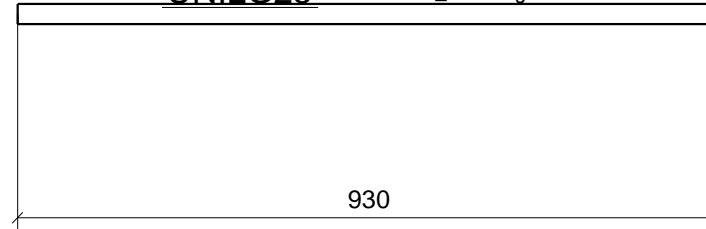
Geprüft / Controllato:  
ST: Sommariva

Datum/Data:

LOTTO	PEZZI
2	1

# 4 SPECIMENS

UNILG25 - UNILG25 - GEWINDE\_M27 x lg. 930.0



MARCA:	Descrizione:	Pezzi	Tratt.Sup.:			
UNILG25	GEWINDESTANGE	4				
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
UNILG25	GEWINDE_M27	10.9	1	930	0.1	4.0
<b>Totale :</b>					<b>0.1</b>	<b>4.0</b>

ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE      NAME - NOME      DATUM - DATA      INDEX

zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

Standard Stahlbau Pichler

Cliente: UNIVERSITA TRENTO

Oggetto: ATTEL

Comessa: 1559

Data: 24.09.2009

Reparto: S

Scala: 1:10

STAHLBAU  
PICHLER GmbH-Srl  
Tel. 0471/065000 - Fax : 0471/065001  
E-MAIL: info@stahlbaupichler.com  
39100 BOZEN/BOLZANO

Disegno:  
**UNILG25**

Gez./Dis.:  
LUKAS H.

Geprüft / Controllato:  
TB: Zelger

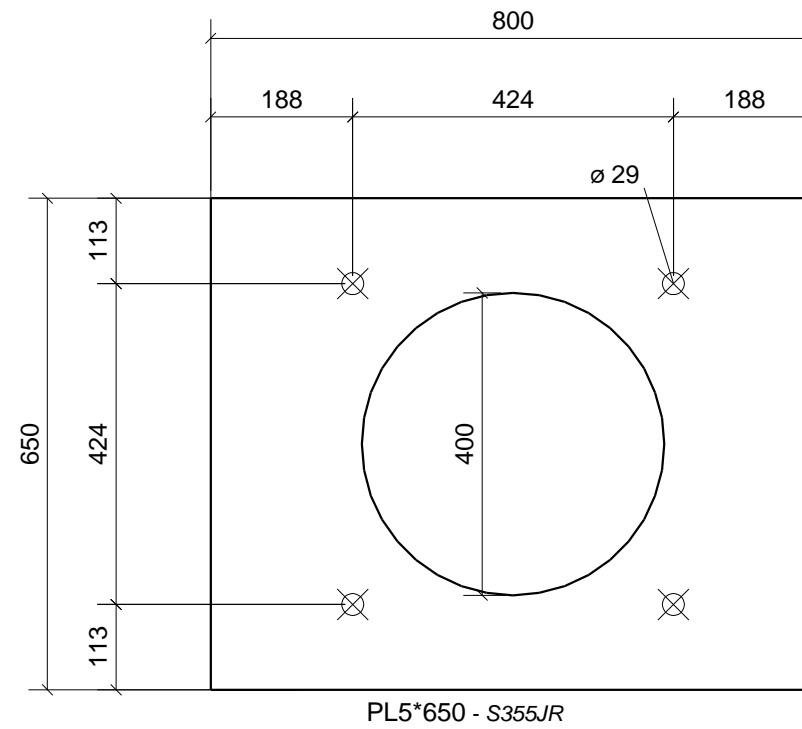
Datum/Data:

Geprüft / Controllato:  
ST: Sommariva

Datum/Data:

LOTTO	PEZZI
2	4

# 1 SPECIMEN



MARCA: <b>UNILG24</b>	Descrizione: PLATTE	Pezzi 1	Tratt.Sup.:			
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
UNILG24	PL5*650	S355JR	1	800	0.8	15.5
Totale :					0.8	15.5

ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE      NAME - NOME      DATUM - DATA      INDEX

zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

Standard Stahlbau Pichler

Cliente: **UNIVERSITA TRENTO**

Oggetto: **ATTEL**

Comessa: **1559**

Data: **27.08.2009**

Reparto: **S**

Scala: **1:10**

Disegno:  
**UNILG24**

Gez./Dis.:  
**LUKAS H.**

Geprüft / Controllato:  
TB: **Zelger**

Datum/Data:

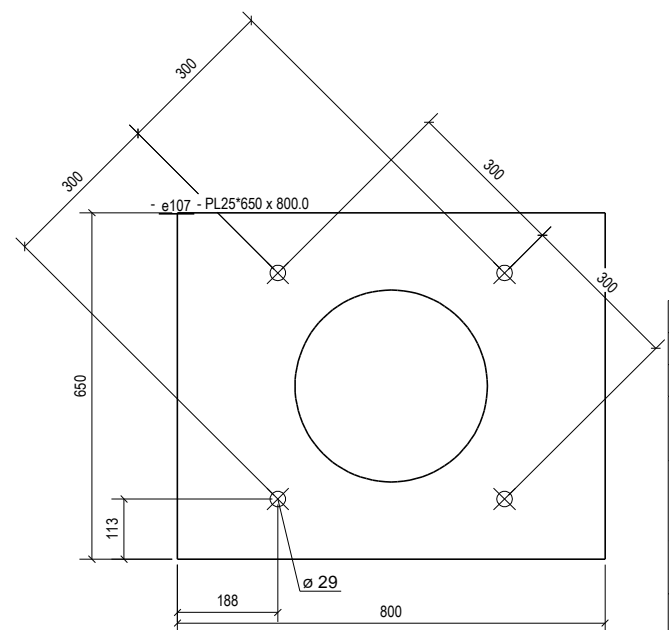
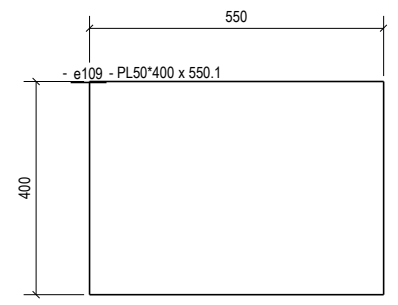
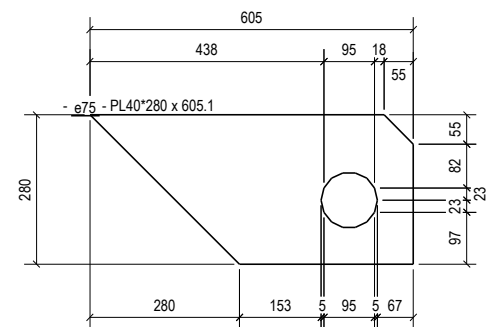
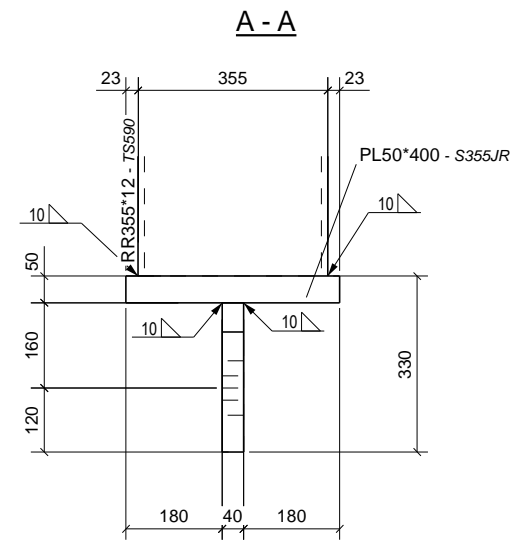
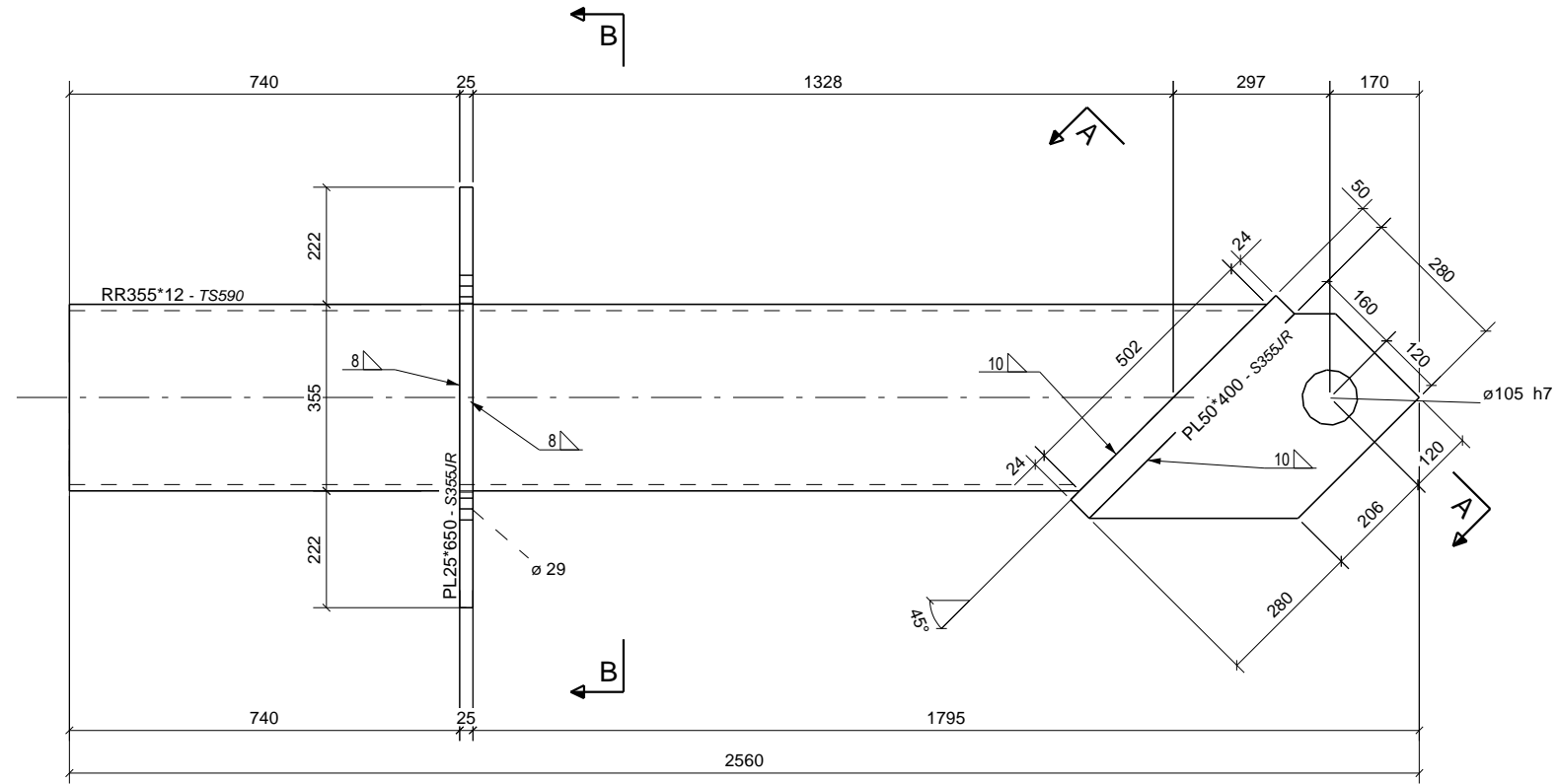
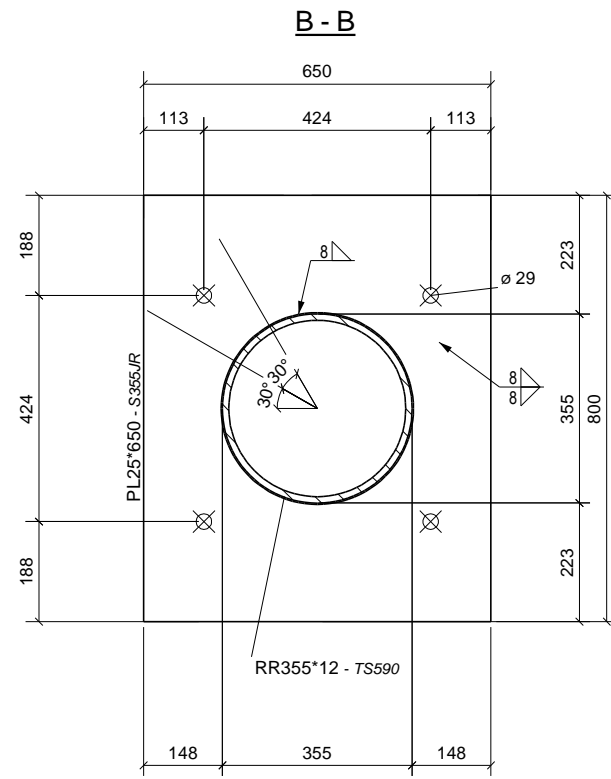
Geprüft / Controllato:  
ST: **Sommariva**

Datum/Data:

LOTTO	PEZZI
2	1



**1 SPECIMEN**



MARCA: UNILG22		Descrizione: HAUPTTRÄGER		Pezzi 1	Tratt. Sup.:		
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso	
e40	RR355*12	TS590	1	2270	2.3	212.3	
e75	PL40*280	S355JR	1	605	0.3	37.8	
e107	PL25*650	S355JR	1	800	0.9	82.1	
e109	PL50*400	S355JR	1	550	0.5	86.4	
<b>Totale :</b>					<b>4.1</b>	<b>418.5</b>	

revisione secondo indicazioni del 18-10-09 **LUKAS H.** 11-11-2009 B

revisione secondo indicazioni **LUKAS H.** 28-09-2009 A

ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE NAME - NOME DATUM - DATA INDEX

zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte : scostamenti ammessi per misure senza tolleranze / saldature : **Standard Stahlbau Pichler**

Cliente: **UNIVERSITA TRENTO**  
 Oggetto: **ATEL**

Comessa: 1559 Data: 13.11.2009  
 Reparto: S Scala: 1:10

Gez./Dis.: **LUKAS H.** Geprüft / Controllato: **TB: Zelger** Datum/Data: Geprüft / Controllato: **ST:Sommariva** Datum/Data:

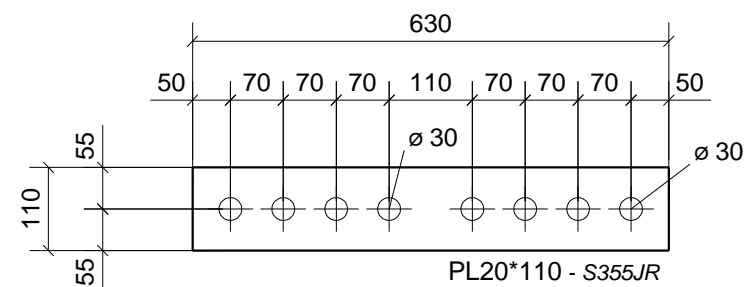
**STAHLBAU PICHLER** GmbH - Srl  
 Tel: 0471 / 065000 - Fax: 0471 / 065001  
 E-MAIL: info@stahlbaupichler.com  
 39100 BOZEN / BOLZANO

Disegno: **UNILG22**

LOTTO	PEZZI
2	1



# 16 SPECIMENS



MARCA: <b>UNILG20</b>	Descrizione: Blech	Pezzi 16	Tratt.Sup.:			
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
UNILG20	PL20*110	S355JR	1	630	0.2	10.9
Totale :					0.2	10.9

ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE NAME - NOME DATUM - DATA INDEX

zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

Standard Stahlbau Pichler

Cliente: **UNIVERSITA TRENTO**

Oggetto: **ATTEL**

Comessa: **1559**

Data: **01.09.2009**

Reparto: **S**

Scala: **1:10**



Gez./Dis.:  
**LUKAS H.**

Geprüft / Controllato:  
TB: **Zelger**

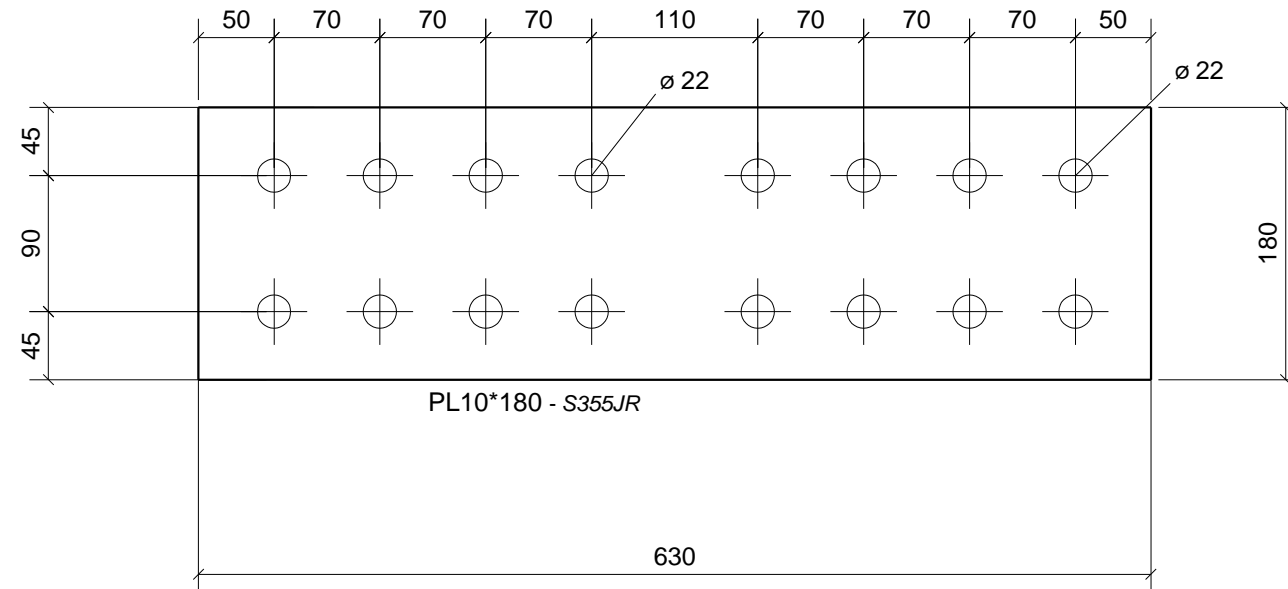
Datum/Data:

Geprüft / Controllato:  
ST: **Sommariva**

Datum/Data:

LOTTO	PEZZI
2	16

# 8 SPECIMENS



MARCA: <b>UNILG19</b>	Descrizione: Blech	Pezzi 8	Tratt.Sup.:			
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
UNILG19	PL10*180	S355JR	1	630	0.2	8.9
Totale :					0.2	8.9

ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE      NAME - NOME      DATUM - DATA      INDEX

zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

Standard Stahlbau Pichler

Cliente: **UNIVERSITA TRENTO**

Oggetto: **ATTEL**

Comessa: **1559**

Data: **01.09.2009**

Reparto: **S**

Scala: **1:5**

**STAHLBAU PICHLER** GmbH-Srl

Tel. 0471/065000 - Fax : 0471/065001  
E-MAIL: info@stahlbaupichler.com  
39100 BOZEN/BOLZANO

Disegno:  
**UNILG19**

Gez./Dis.:  
**LUKAS H.**

Geprüft / Controllato:  
**TB: Zelger**

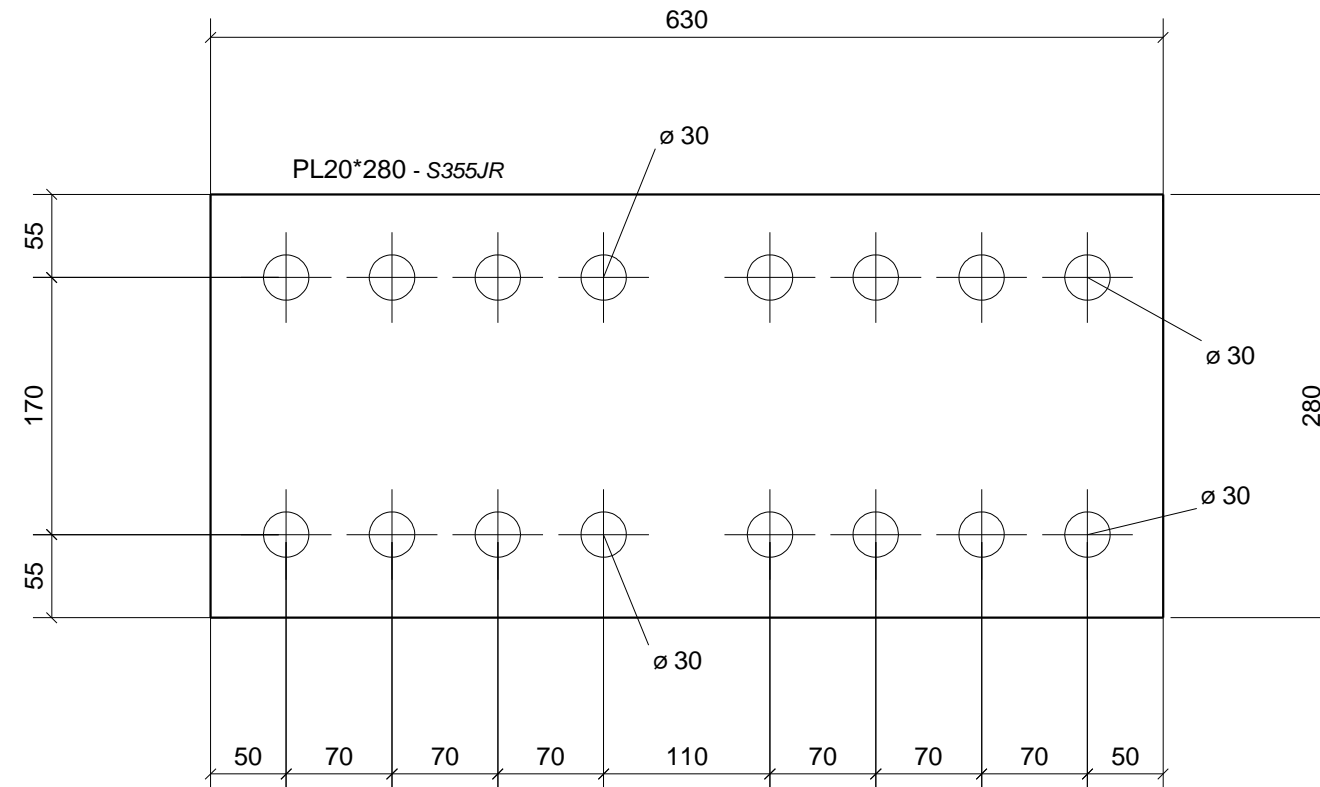
Datum/Data:

Geprüft / Controllato:  
**ST: Sommariva**

Datum/Data:

LOTTO	PEZZI
2	8

# 8 SPECIMENS



MARCA: <b>UNILG18</b>	Descrizione: Blech	Pezzi 8	Tratt.Sup.:			
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
UNILG18	PL20*280	S355JR	1	630	0.4	27.7
Totale :					0.4	27.7

ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE      NAME - NOME      DATUM - DATA      INDEX

zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

Standard Stahlbau Pichler

Cliente: **UNIVERSITA TRENTO**

Oggetto: **ATTEL**

Comessa: **1559**

Data: **01.09.2009**

Reparto: **S**

Scala: **1:5**

**STAHLBAU PICHLER** GmbH-Srl

Tel.0471/065000 - Fax : 0471/065001  
E-MAIL: info@stahlbaupichler.com  
39100 BOZEN/BOLZANO

**Disegno:**  
**UNILG18**

Gez./Dis.:  
**LUKAS H.**

Geprüft / Controllato:  
**TB: Zelger**

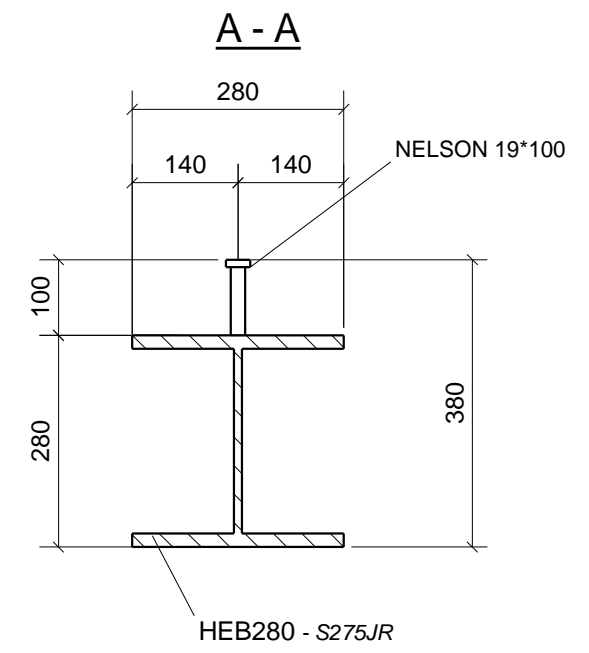
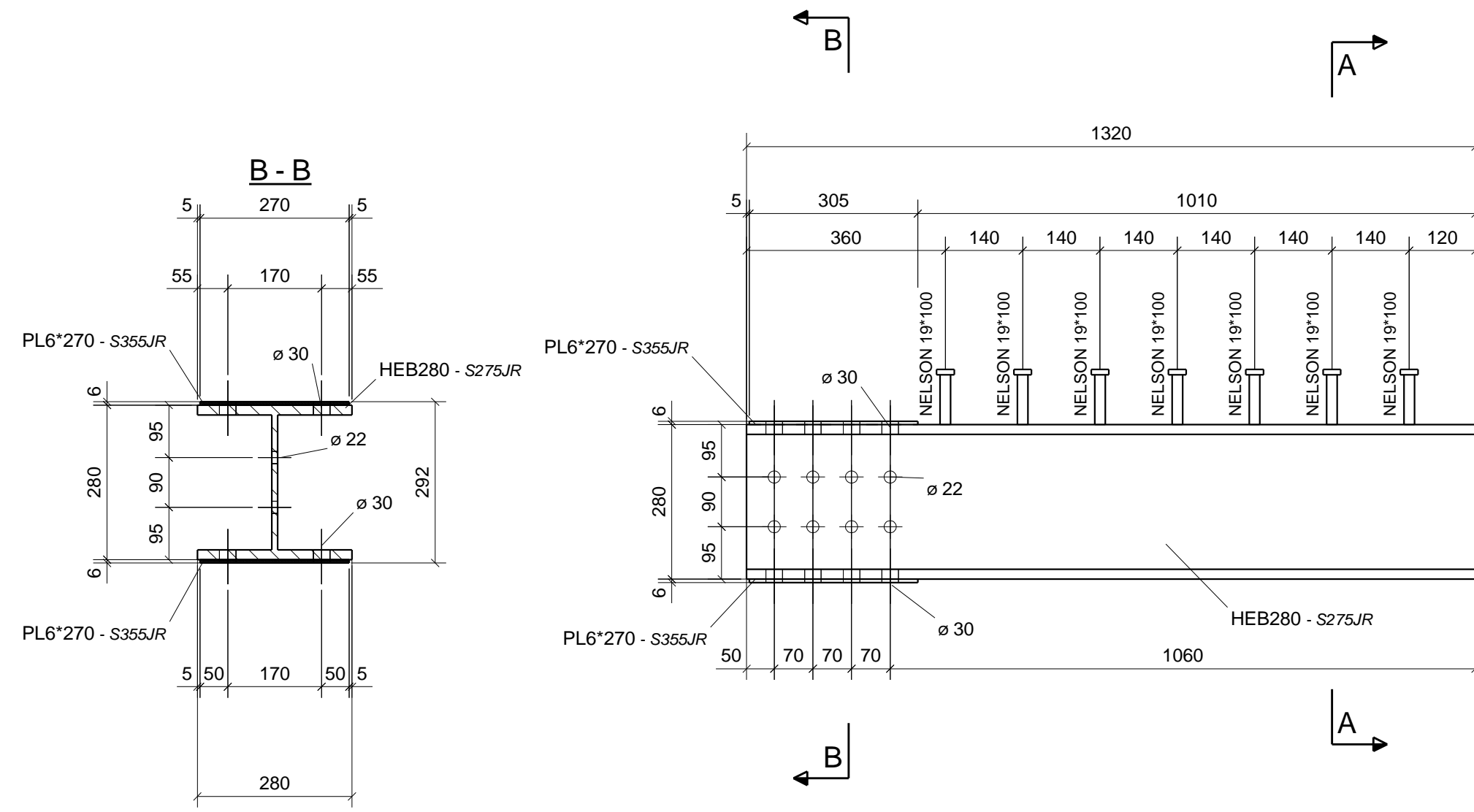
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Geprüft / Controllato:  
**ST: Sommariva**

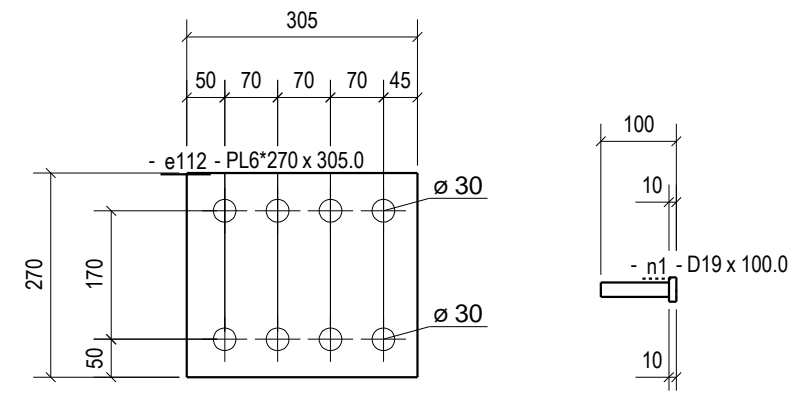
Datum/Data:

LOTTO	PEZZI
2	8

# 4 SPECIMENS



MARCA:	Descrizione:	Pezzi	Tratt.Sup.:			
UNILG17	UNILG	4				
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
e93	HEB280	S275JR	1	1320	2.1	135.7
e112	PL6*270	S355JR	2	305	0.2	3.9
n1	D19	S355JR	7	100	0.0	0.2
<b>Totale :</b>					<b>2.5</b>	<b>145.2</b>



LOTTO	PEZZI
2	4

ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE      NAME - NOME      DATUM - DATA      INDEX

zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

**Standard Stahlbau Pichler**

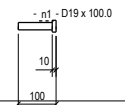
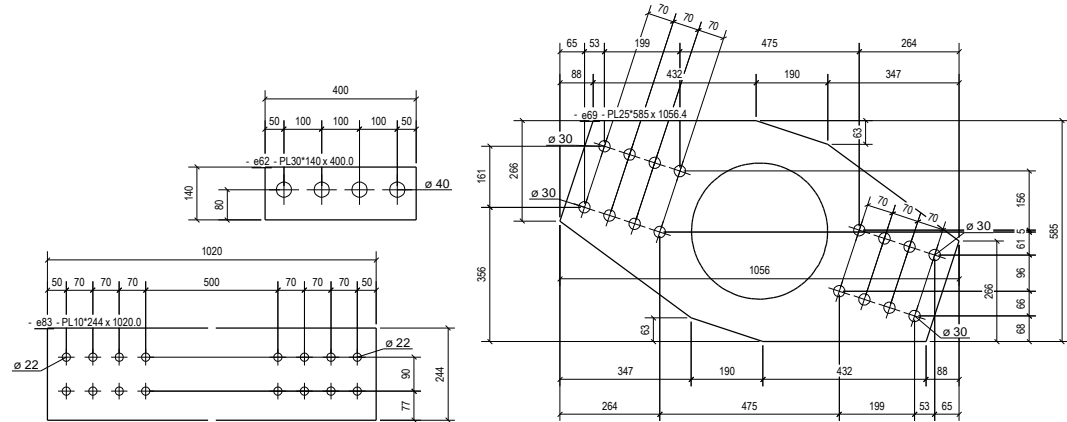
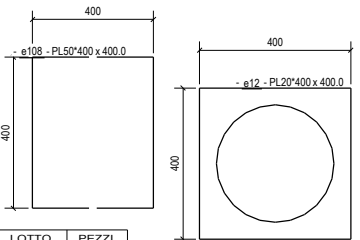
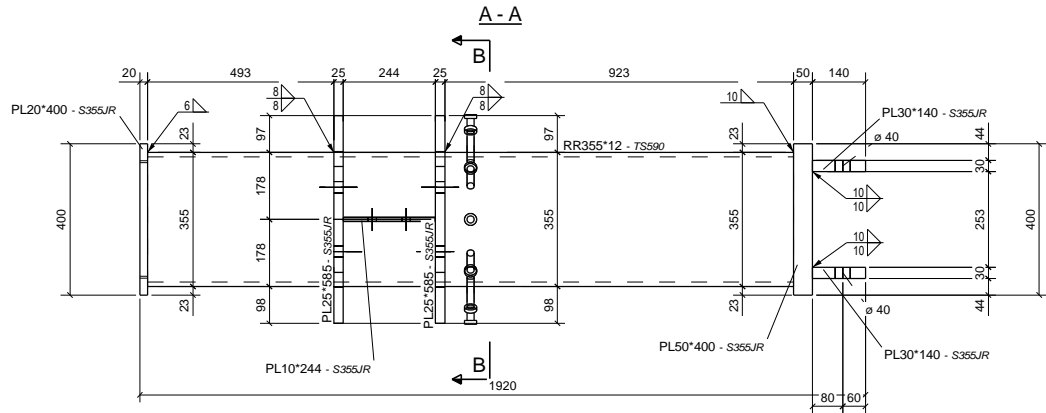
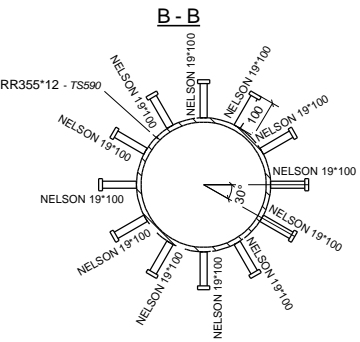
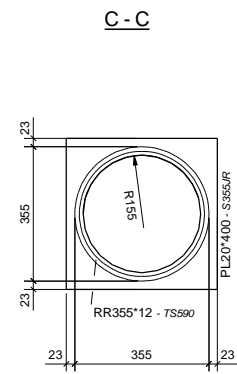
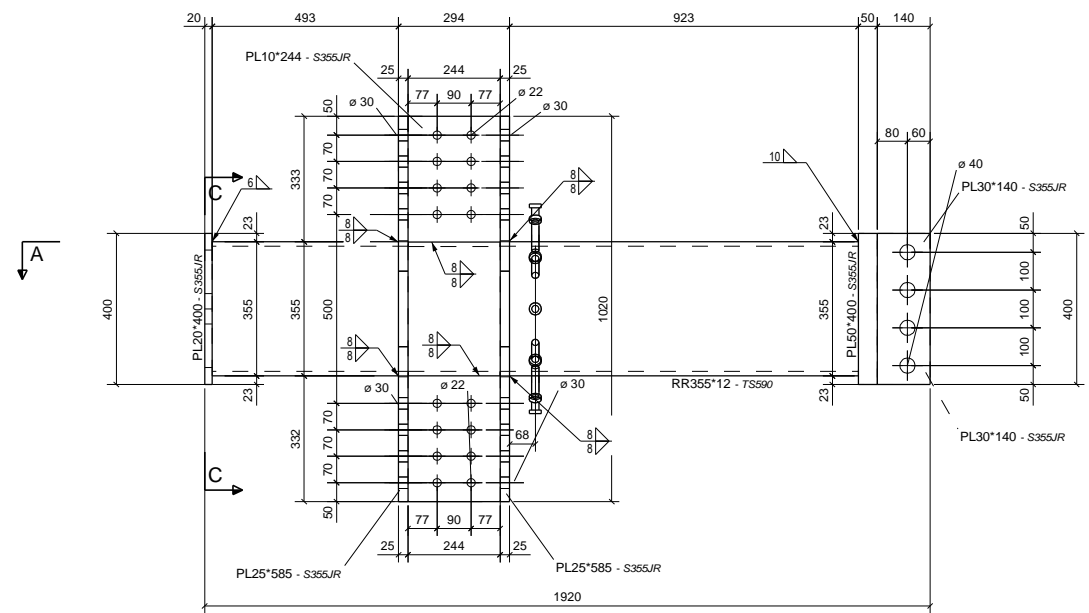
Cliente: **UNIVERSITA TRENTO**  
 Oggetto: **ATTEL**

Comessa: **1559**      Data: **01.09.2009**  
 Reparto: **S**      Scala: **1:10**

Gez./Dis.: **LUKAS H.**      Geprüft / Controllato: **TB: Zelger**      Datum/Data:      Geprüft / Controllato: **ST: Sommariva**      Datum/Data:

STAHLBAU PICHLER  
Tel.0471/065000 - Fax : 0471/065001  
E-MAIL: info@stahlbaupichler.com  
39100 BOZEN/BOLZANO

**UNILG17**



MARCA: UNILG16		Descrizione: HAUPTRÄGER		Pezzi 2		Tratt.Sup.:	
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso	
e4	RR355*12	TS590	1	1710	1.9	172.9	
e12	PL20*400	S355JR	1	400	0.2	13.3	
e62	PL30*140	S355JR	2	400	0.1	13.2	
e69	PL25*585	S355JR	2	1056	0.8	68.4	
e83	PL10*244	S355JR	1	1020	0.5	19.5	
e108	PL50*400	S355JR	1	400	0.4	62.8	
n1	D19	S355JR	12	100	0.0	0.3	
				<b>Totale :</b>	<b>5.0</b>	<b>434.9</b>	

revisions secondo indicazioni

ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE

zuliässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

Cliente: UNIVERSITA TRENTO  
Oggetto: ATTEL

Comessa: 1559  
Reparto: S

Data: 12.11.2009  
Scala: 1:10

Geprüft / Controllato: TB: Zelger  
Datum/Data:

Standard Stahlbau Pichler

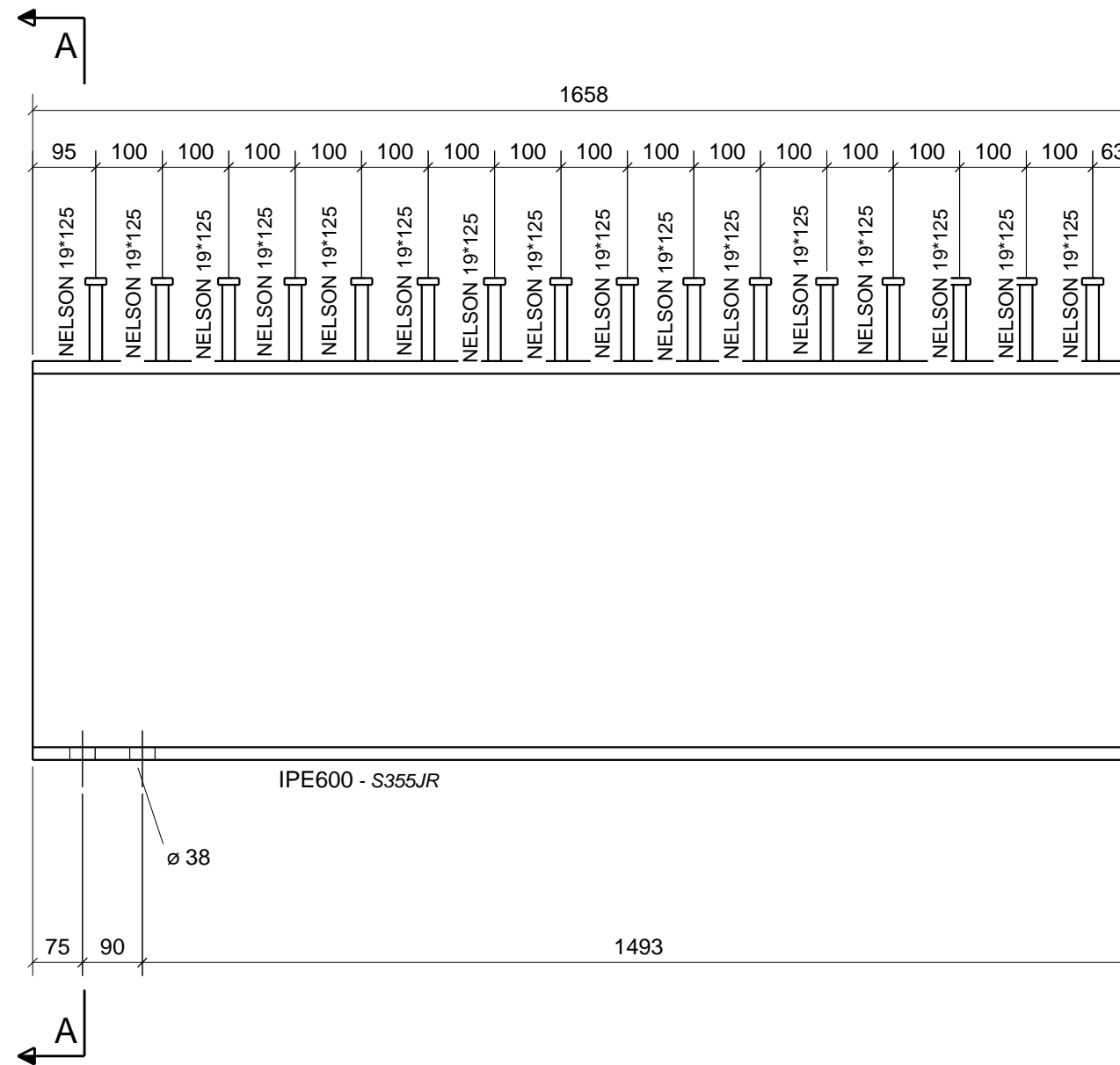
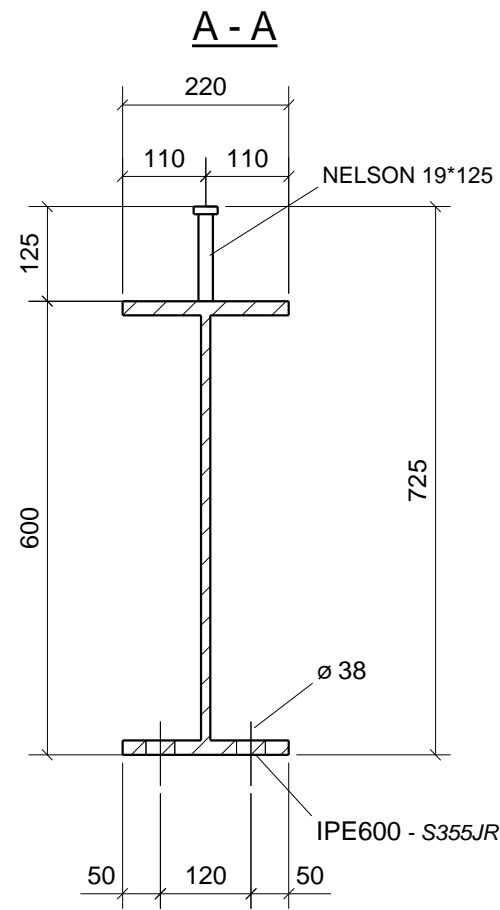
STAHLSBAU PICHLER

Disegno: UNILG16

Geprüft / Controllato: ST: Sommariva  
Datum/Data:

LOTTO	PEZZI
2	2

**4 SPECIMENS**



MARCA: <b>UNILG15</b>	Descrizione: UNILG	Pezzi 4	Tratt.Sup.:			
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
e104	IPE600	S355JR	1	1658	3.3	203.0
n2	D19	S355JR	16	125	0.0	0.3
<b>Totale :</b>					<b>3.5</b>	<b>207.7</b>

ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE      NAME - NOME      DATUM - DATA      INDEX

zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

**Standard Stahlbau Pichler**

Cliente: **UNIVERSITA TRENTO**

Oggetto: **ATTEL**

Comessa: **1559**

Data: **27.08.2009**

Reparto: **S**

Scala: **1:10**

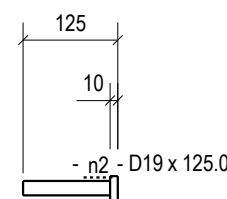
Gez./Dis.:  
**LUKAS H.**

Geprüft / Controllato:  
TB: **Zelger**

Datum/Data:

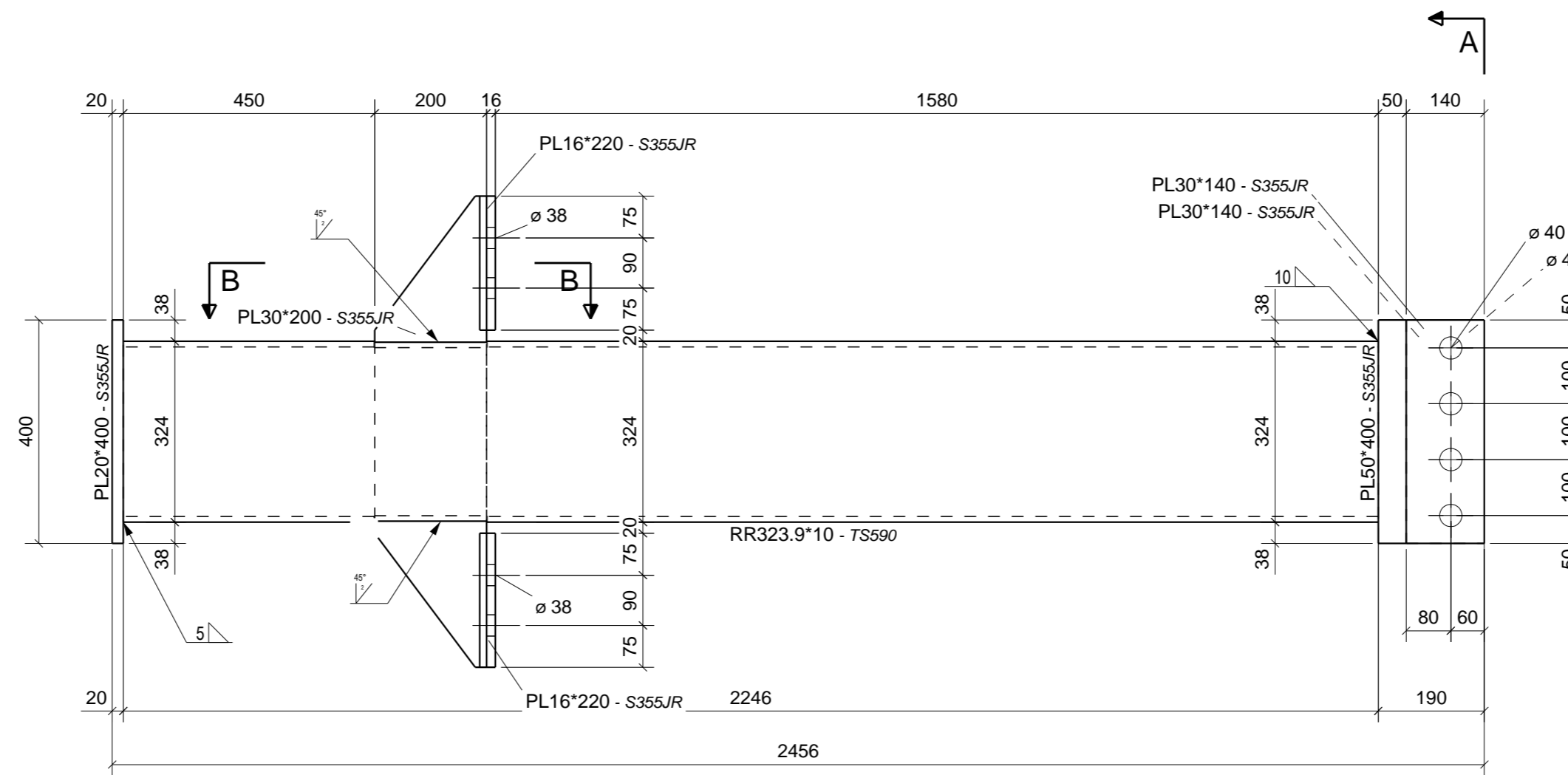
Geprüft / Controllato:  
ST: **Sommariva**

Datum/Data:

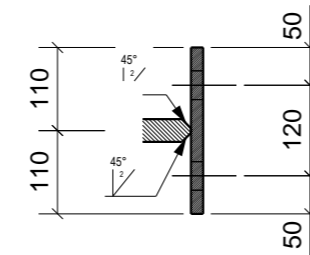


LOTTO	PEZZI
2	4

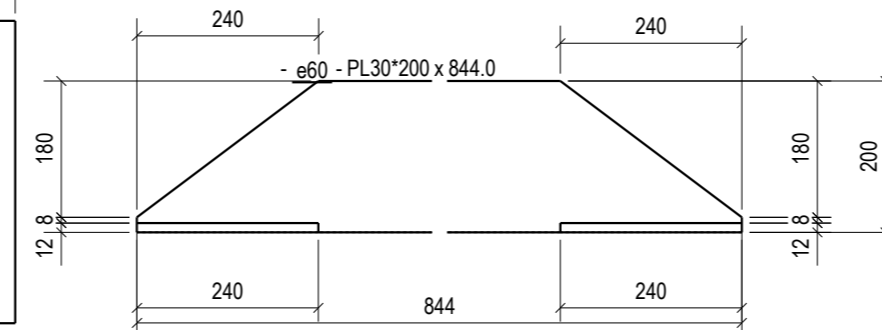
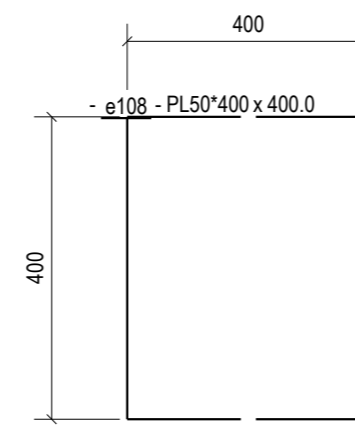
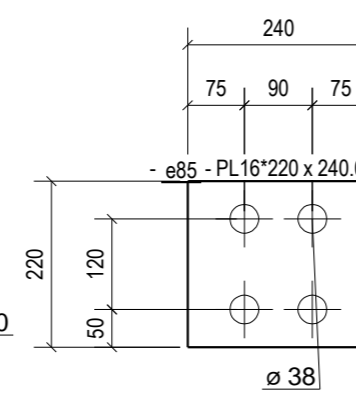
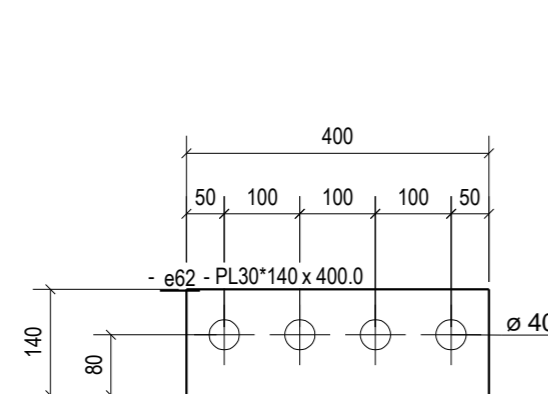
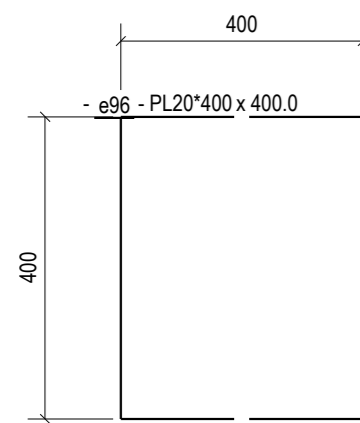
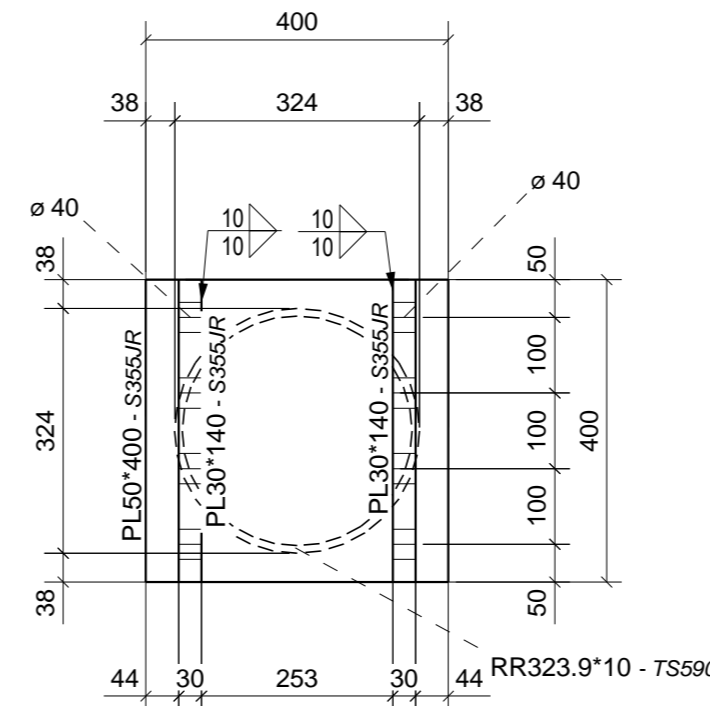
**2 SPECIMENS**



**B - B**



**A - A**

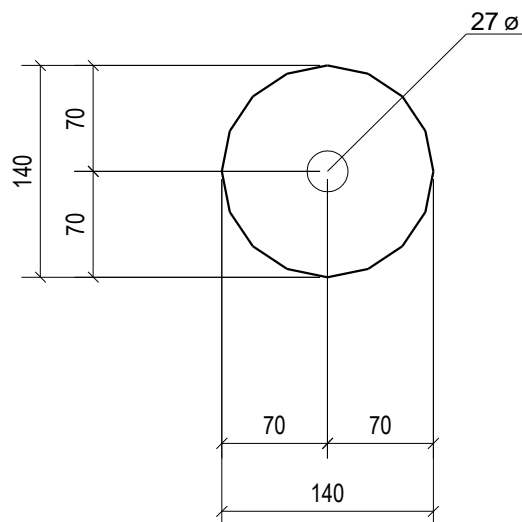


MARCA: UNILG14		Descrizione: HAUPTTRÄGER		Pezzi	Tratt.Sup.:		
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso	
e60	PL30*200	S355JR	1	844	0.3	29.0	
e62	PL30*140	S355JR	2	400	0.1	13.2	
e85	PL16*220	S355JR	2	240	0.1	6.6	
e86	RR323.9*10	TS590	1	2246	2.3	173.9	
e96	PL20*400	S355JR	1	400	0.4	25.1	
e108	PL50*400	S355JR	1	400	0.4	62.8	
<b>Totale :</b>					<b>3.9</b>	<b>330.5</b>	

revisone secondo indicazioni		LUKAS H.		28-09-2009	A
ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE		NAME - NOME		DATUM - DATA	INDEX
zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte : scostamenti ammessi per misure senza tolleranze / saldature :				<b>Standard Stahlbau Pichler</b>	
Cliente: UNIVERSITA TRENTO		Data: 28.09.2009			
Oggetto: ATTEL		Scala: 1:10			
Comessa: 1559		Reparto: S		Disegno: UNILG14	
Gez./Dis.: LUKAS H.		Geprüft / Controllato: TB: Zelger		Datum/Data: ST:Sommariva	

LOTTO	PEZZI
2	2

# 16 SPECIMEN



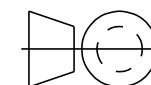
LOTTO	PEZZI
2	16

Quantità	per Marca
16	UNILG13

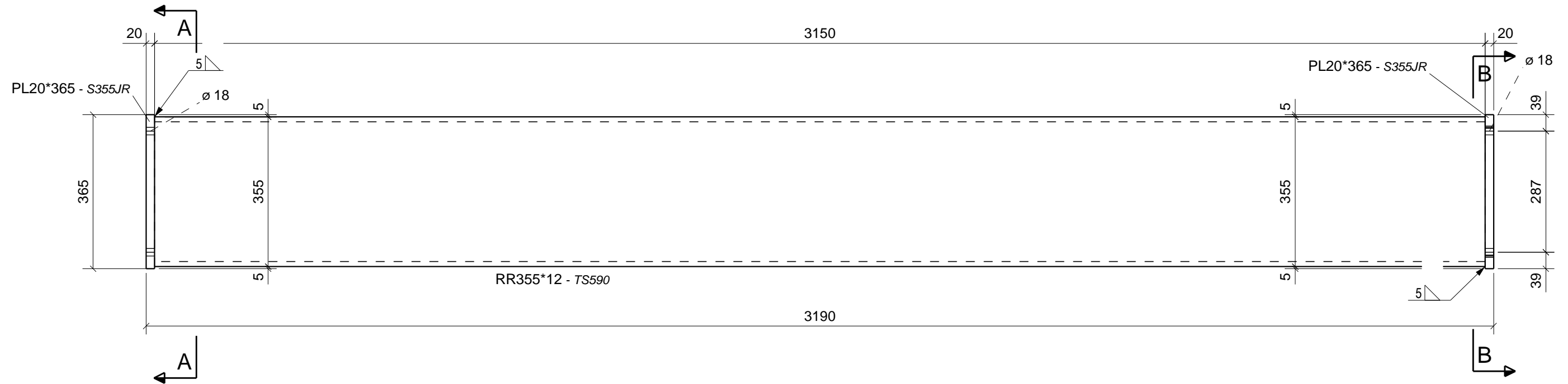
Cliente: UNIVERSITA TRENTO	Comessa: 1559	Reparto: S	Disegno: PLATTE
Descrizione: PLATTE	Profilo: PL25*140	Quantità: 16	Peso: 2.9
Lunghezza: 140	Posizione: UNILG13	Data: 24.09.2009	Modifica:
Superficie: 0.04			Rev:
Materiale: S355JR			



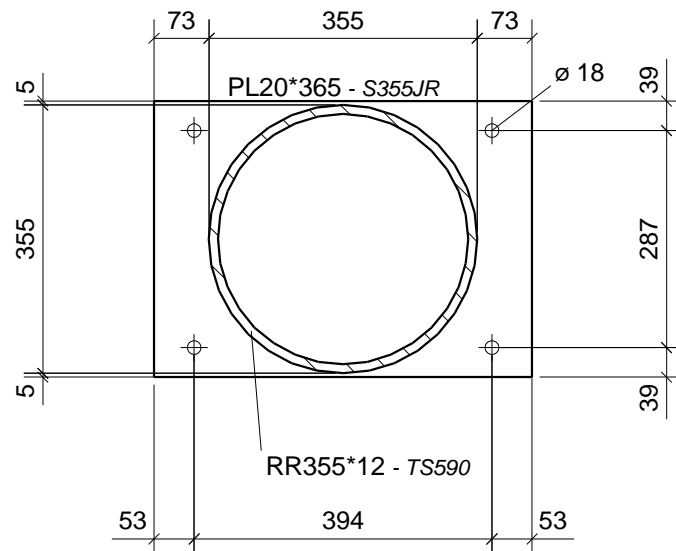
**STAHLBAU  
PICHLER** GmbH-Srl  
Tel.0471/065000 - Fax : 0471/065001  
E-MAIL: info@stahlbaupichler.com  
39100 BOZEN/BOLZANO



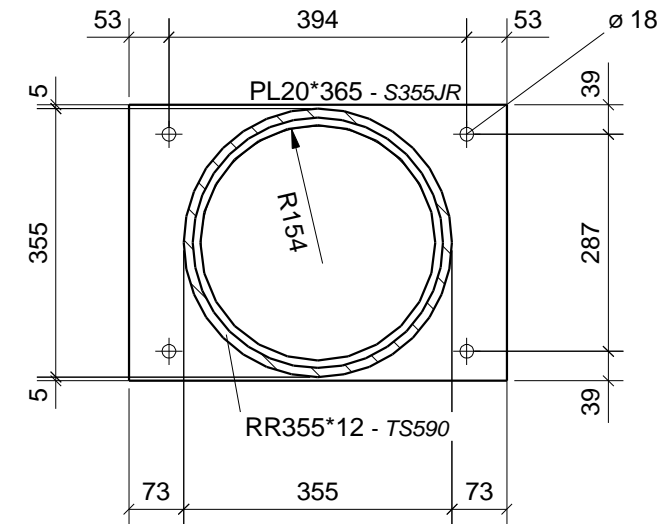
# 1 SPECIMEN



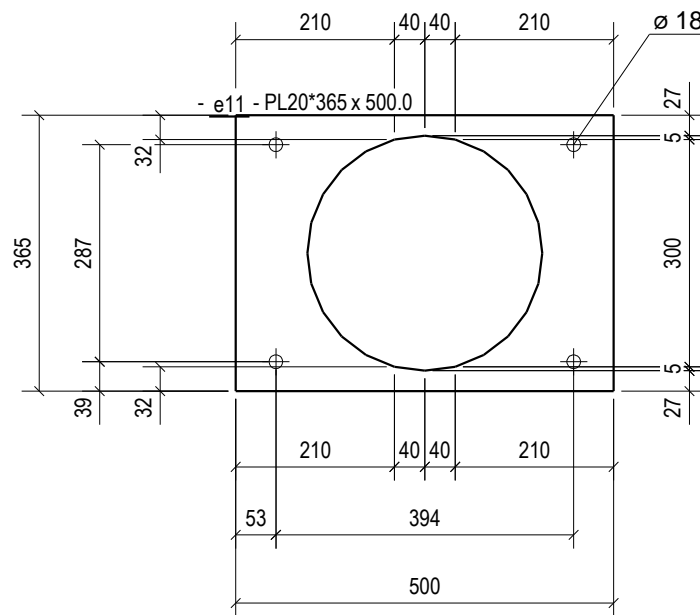
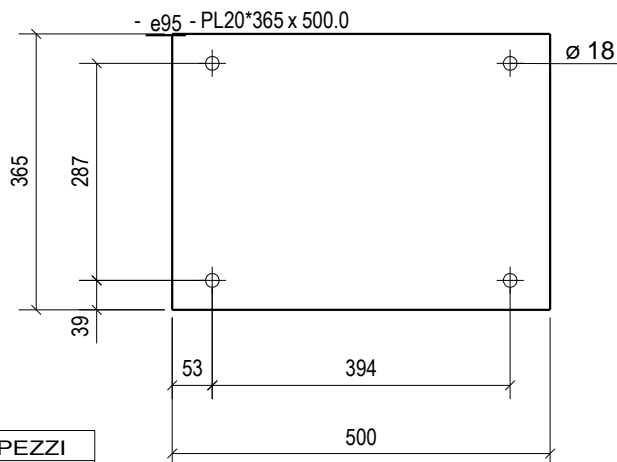
**A - A**




**B - B**



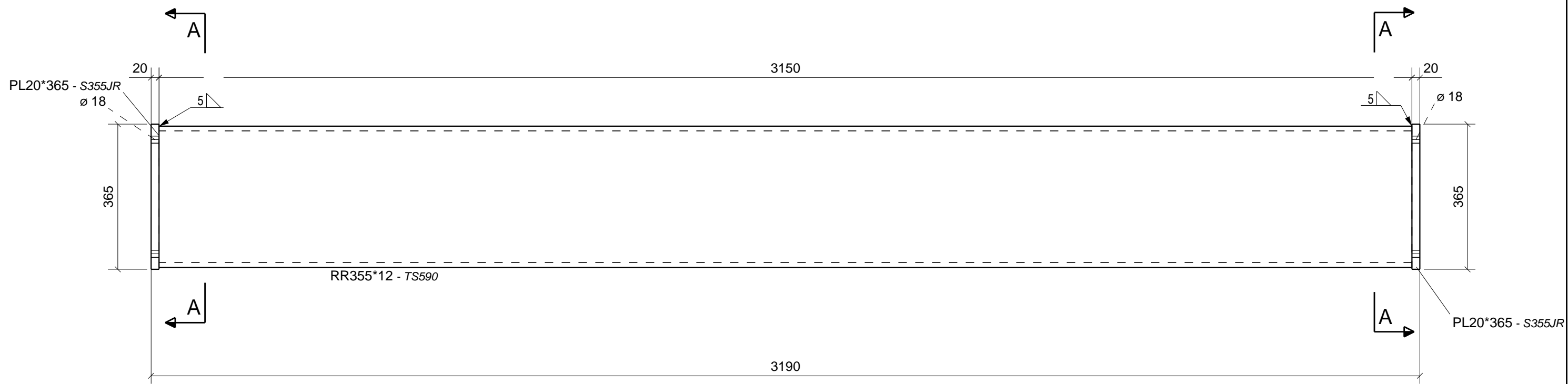
MARCA:	Descrizione:	Pezzi	Tratt.Sup.:			
<b>UNILG12</b>	<b>HAUPTTRÄGER</b>	<b>1</b>				
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
e11	PL20*365	S355JR	1	500	0.3	16.9
e34	RR355*12	TS590	1	3150	3.4	317.7
e95	PL20*365	S355JR	1	500	0.4	28.7
<b>Totale :</b>					<b>4.1</b>	<b>363.2</b>



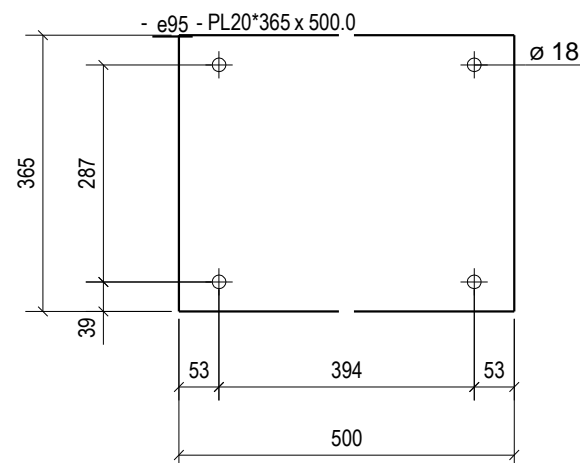
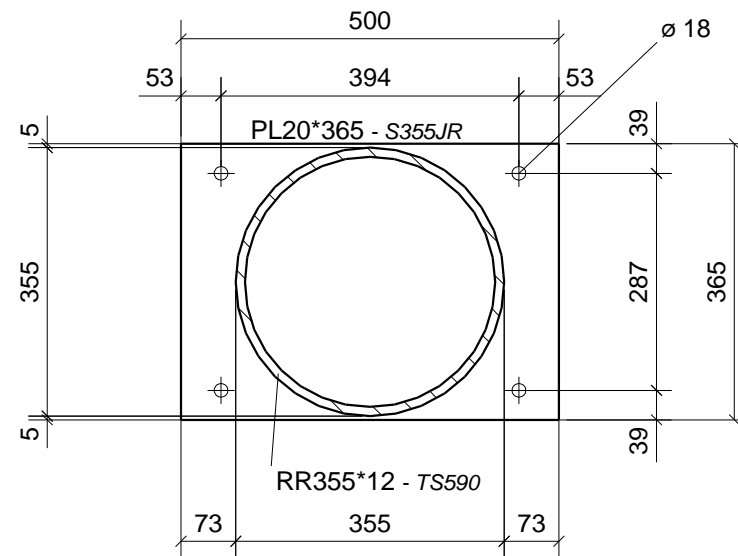
LOTTO	PEZZI
2	1

revisione secondo indicazioni		LUKAS H.		28-09-2009		A	
ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE		NAME - NOME		DATUM - DATA		INDEX	
zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte : scostamenti ammessi per misure senza tolleranze / saldature :				<b>Standard Stahlbau Pichler</b>			
Cliente: UNIVERSITA TRENTO		Oggetto: ATTEL				 Disegno: <b>UNILG12</b>	
Comessa: 1559		Data: 24.09.2009					
Reparto: S		Scala: 1:10		Geprüft / Controllato: ST: Sommariva		Datum/Data:	
Gez./Dis.: LUKAS H.		Geprüft / Controllato: TB: Zelger		Datum/Data:		Datum/Data:	

# 1 SPECIMEN



## A - A



MARCA: <b>UNILG11</b>	Descrizione: <b>HAUPTTRÄGER</b>	Pezzi <b>1</b>	Tratt.Sup.:			
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
e34	RR355*12	TS590	1	3150	3.4	317.7
e95	PL20*365	S355JR	2	500	0.4	28.7
<b>Totale :</b>					<b>4.2</b>	<b>375.0</b>

ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE NAME - NOME DATUM - DATA INDEX

zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

Standard Stahlbau Pichler

Cliente: **UNIVERSITA TRENTO**

Oggetto: **ATTEL**

Comessa: **1559**

Data: **31.08.2009**

Reparto: **S**

Scala: **1:10**

Gez./Dis.:  
**LUKAS H.**

Geprüft / Controllato:  
TB: **Zelger**

Datum/Data:

Geprüft / Controllato:  
ST: **Sommariva**

Datum/Data:

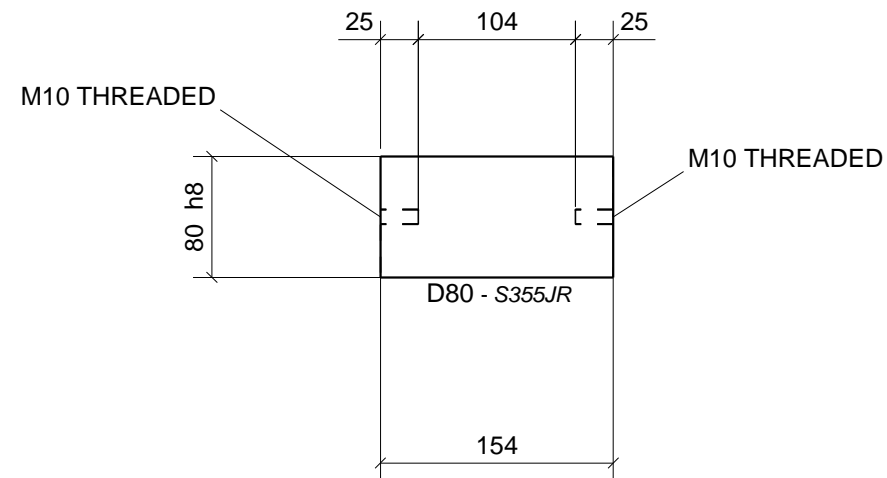
**STAHLBAU PICHLER** GmbH-Srl  
Tel. 0471/065000 - Fax : 0471/065001  
E-MAIL: info@stahlbaupichler.com  
39100 BOZEN/BOLZANO

Disegno:  
**UNILG11**

LOTTO	PEZZI
2	1



**2 SPECIMENS**


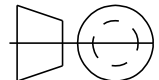



MARCA: <b>HINGE3</b>	Descrizione: PROFIL	Pezzi 2	Tratt.Sup.:			
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
HINGE3	D80	S355JR	1	154	0.1	6.0
Totale :					0.1	6.0

revisione secondo indicazioni			LUKAS H.	28-09-2009	A
ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE			NAME - NOME	DATUM - DATA	INDEX

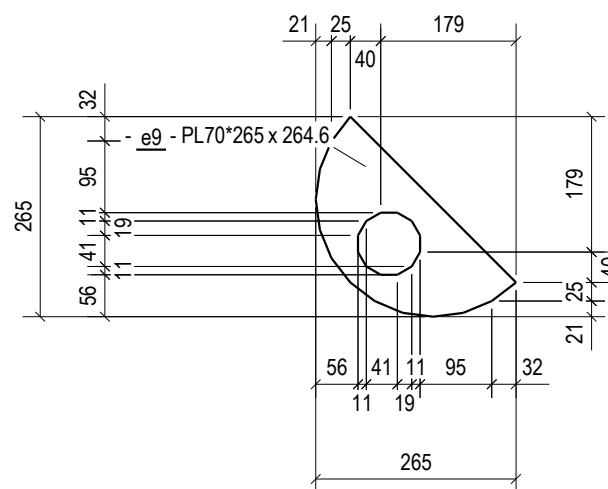
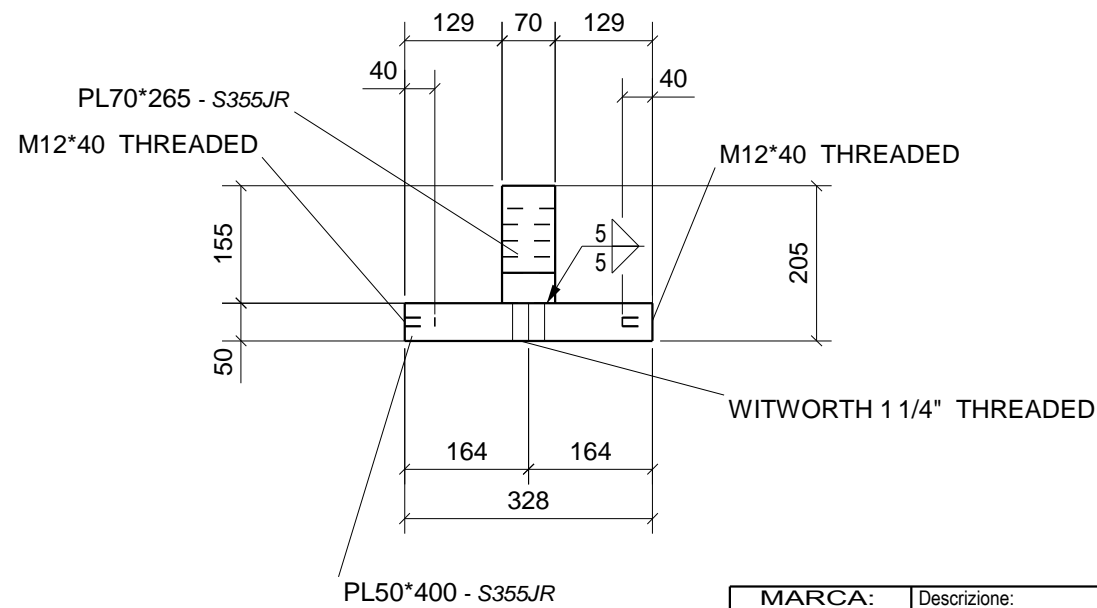
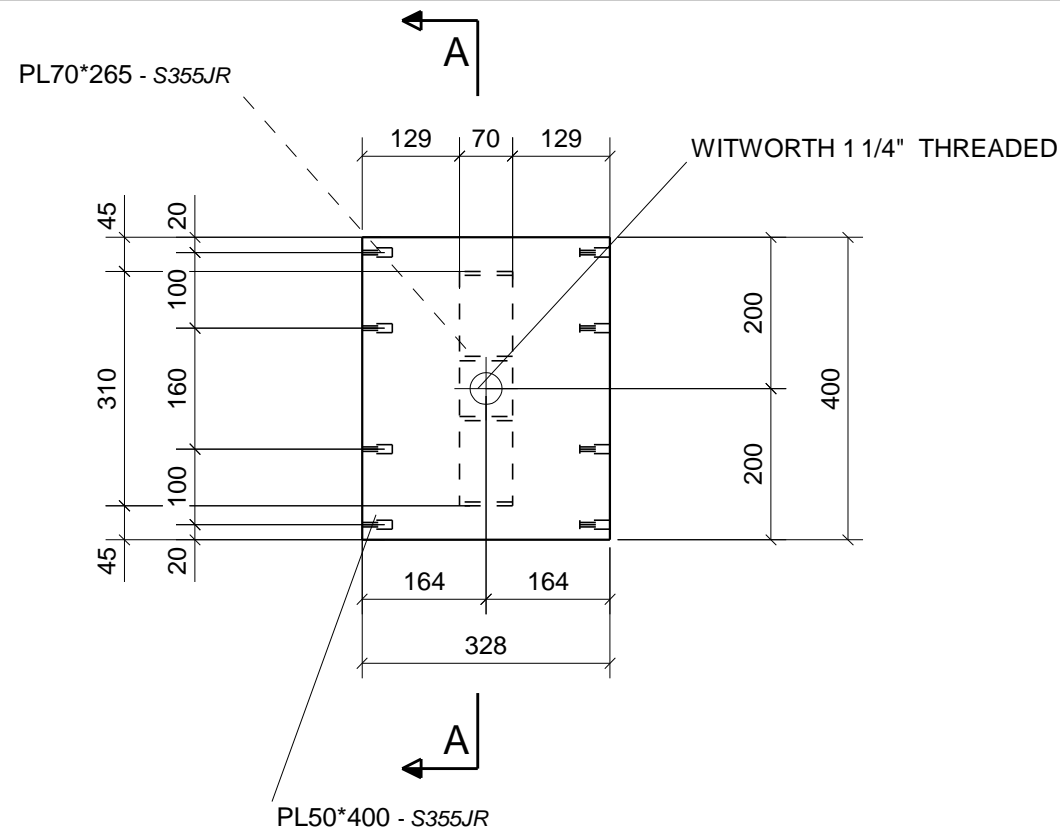
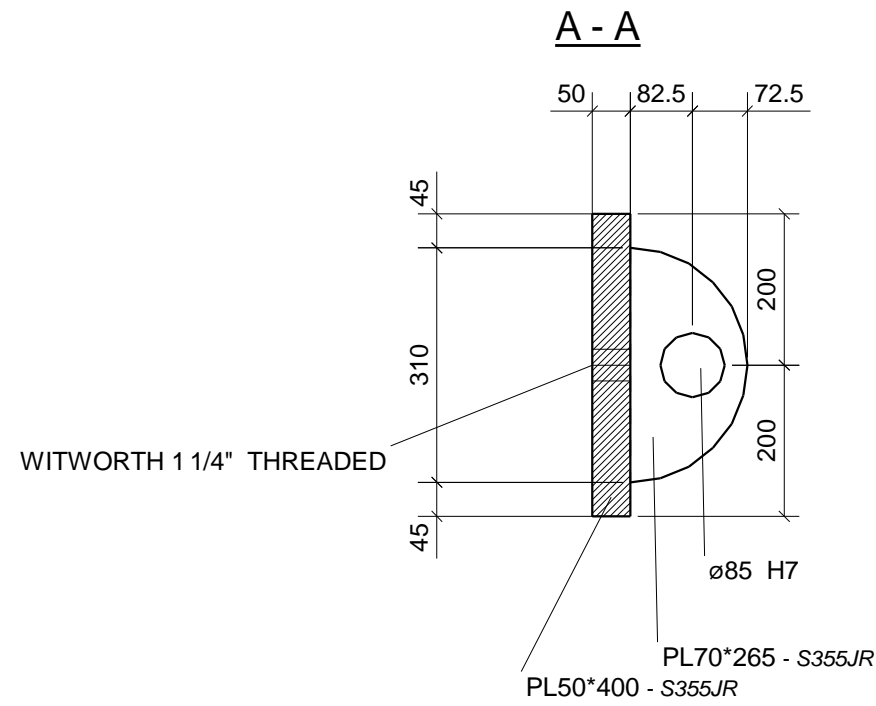
zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

Standard Stahlbau Pichler

Cliente: UNIVERSITA TRENTO		
Oggetto: ATTEL		
Comessa: 1559	Data: 13.11.2009	
Reparto: S	Scala: 1:5	
Gez./Dis.: LUKAS H.	Geprüft / Controllato: TB: Zelger	Datum/Data:
	Geprüft / Controllato: ST: Sommariva	Datum/Data:

LOTTO	PEZZI
2	2

**2 SPECIMENS**

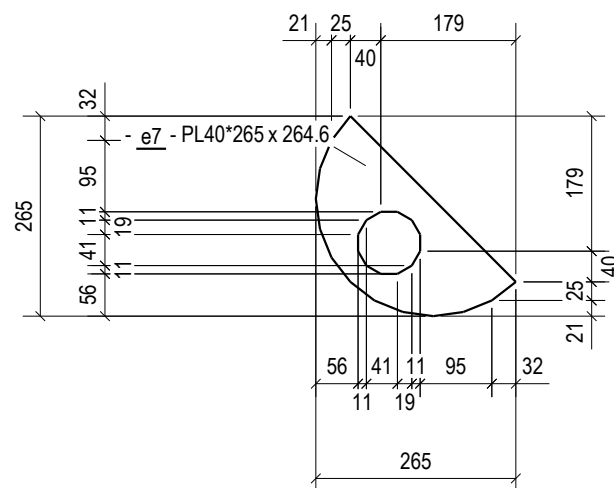
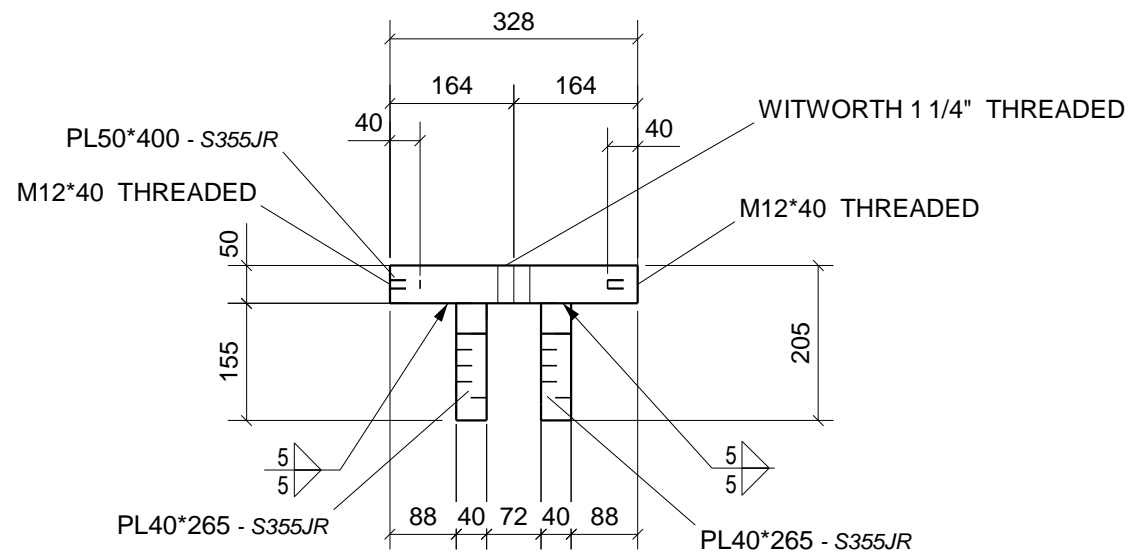
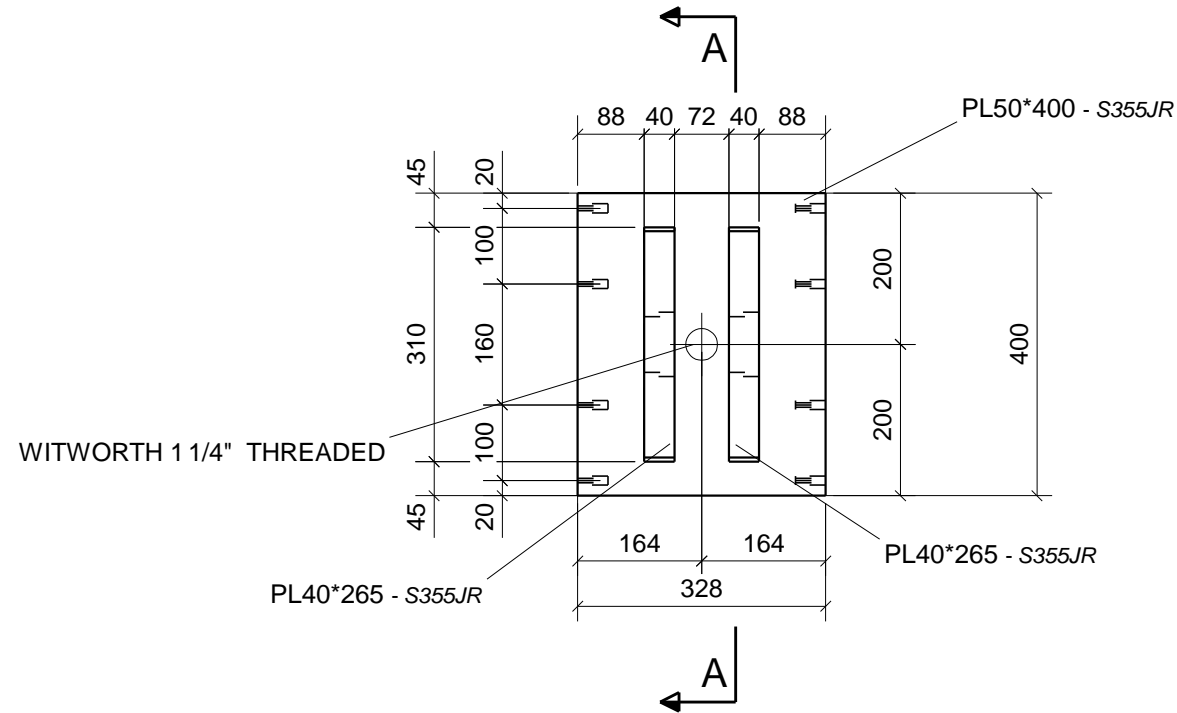
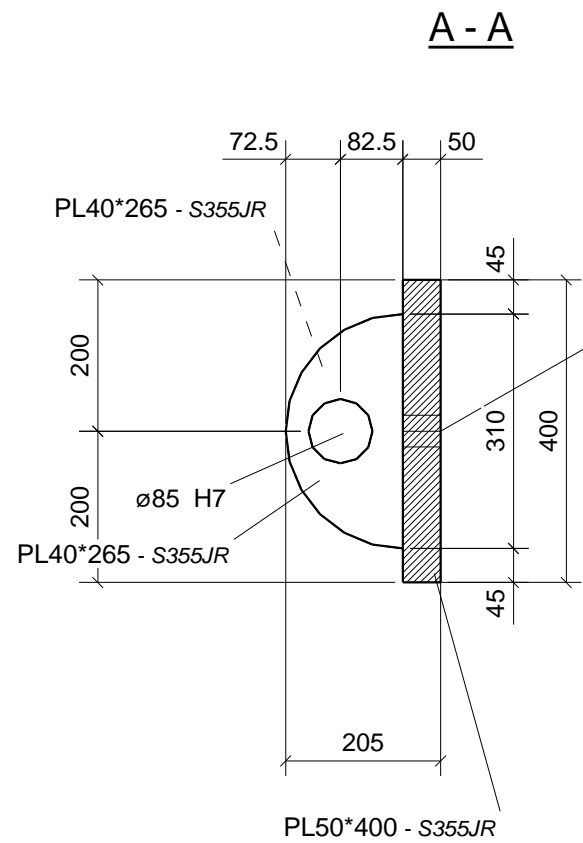


MARCA: <b>HINGE2</b>		Descrizione: <b>PLATTE</b>		Pezzi: <b>2</b>		Tratt.Sup.:	
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso	
e8	PL50*400	S355JR	1	328	0.3	51.2	
e9	PL70*265	S355JR	1	265	0.1	17.5	
Totale :					0.5	68.7	

revisione secondo indicazioni			LUKAS H.		28-09-2009		A	
ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE			NAME - NOME		DATUM - DATA		INDEX	
zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte : scostamenti ammessi per misure senza tolleranze / saldature :					<b>Standard Stahlbau Pichler</b>			
Cliente: <b>UNIVERSITA TRENTO</b>			Oggetto: <b>ATTEL</b>					
Comessa: <b>1559</b>		Data: <b>13.11.2009</b>						
Reparto: <b>S</b>		Scala: <b>1:10</b>		Geprüft / Controllato: <b>TB: Zelger</b>		Datum/Data:		
Gez./Dis.: <b>LUKAS H.</b>		Geprüft / Controllato: <b>ST: Sommariva</b>		Datum/Data:		Disegno: <b>HINGE2</b>		

LOTTO	PEZZI
2	2

**2 SPECIMENS**



MARCA: <b>HINGE1</b>	Descrizione: <b>PLATTE</b>	Pezzi <b>2</b>	Tratt.Sup.:			
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
e7	PL40*265	S355JR	2	265	0.1	10.0
e8	PL50*400	S355JR	1	328	0.3	51.2
Totale :					0.6	71.3

revisione secondo indicazioni		LUKAS H.	28-09-2009	A
ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE		NAME - NOME	DATUM - DATA	INDEX

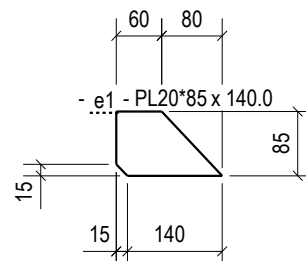
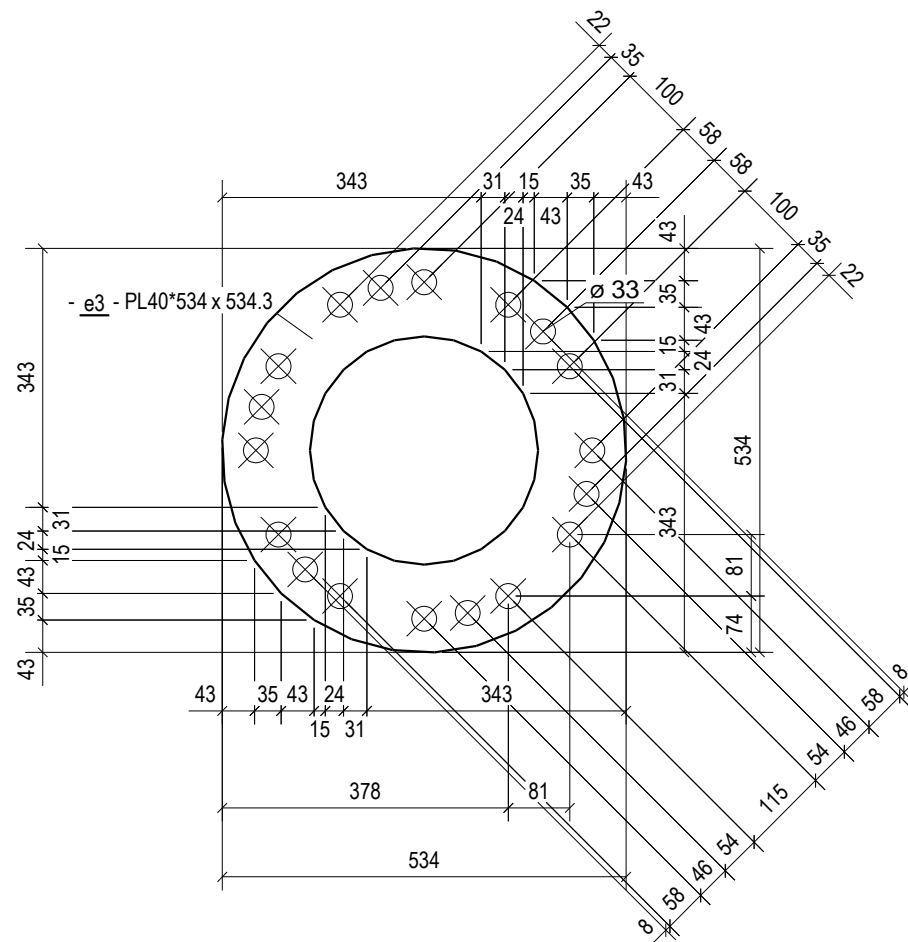
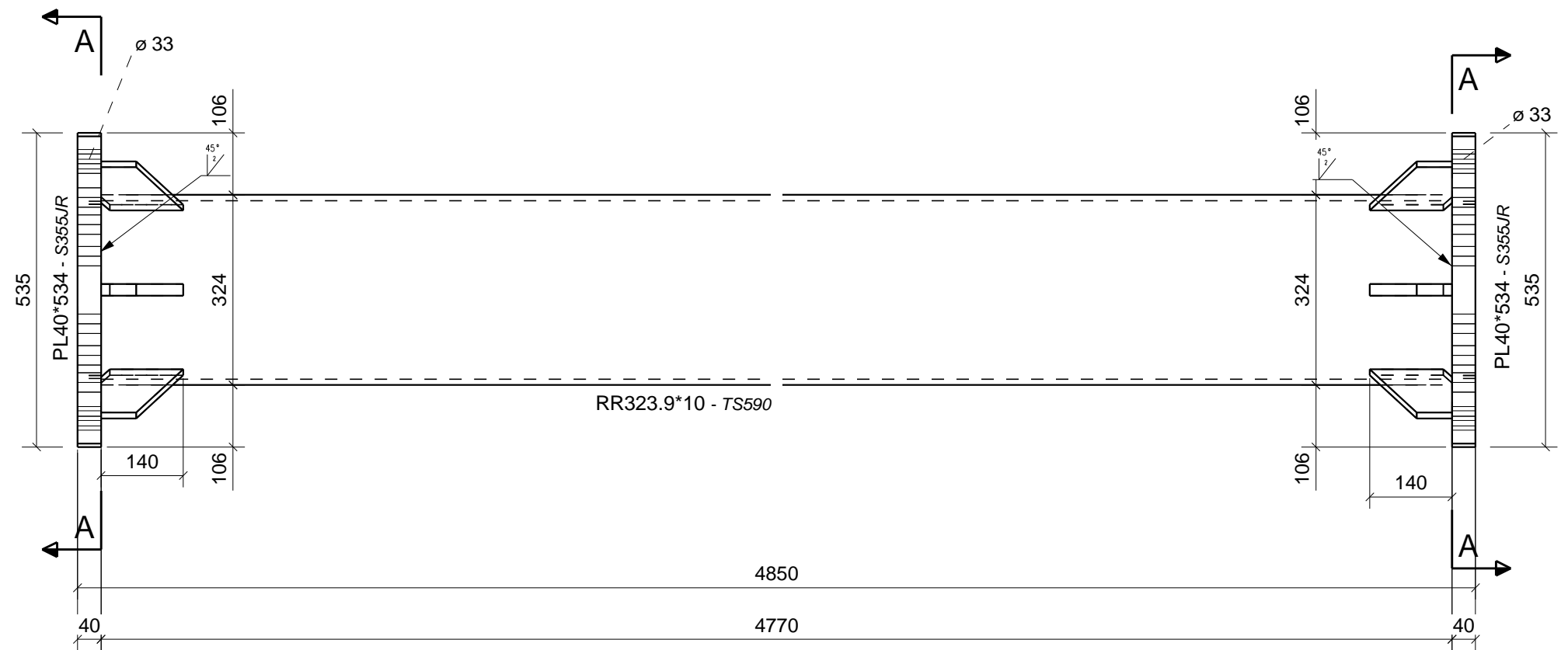
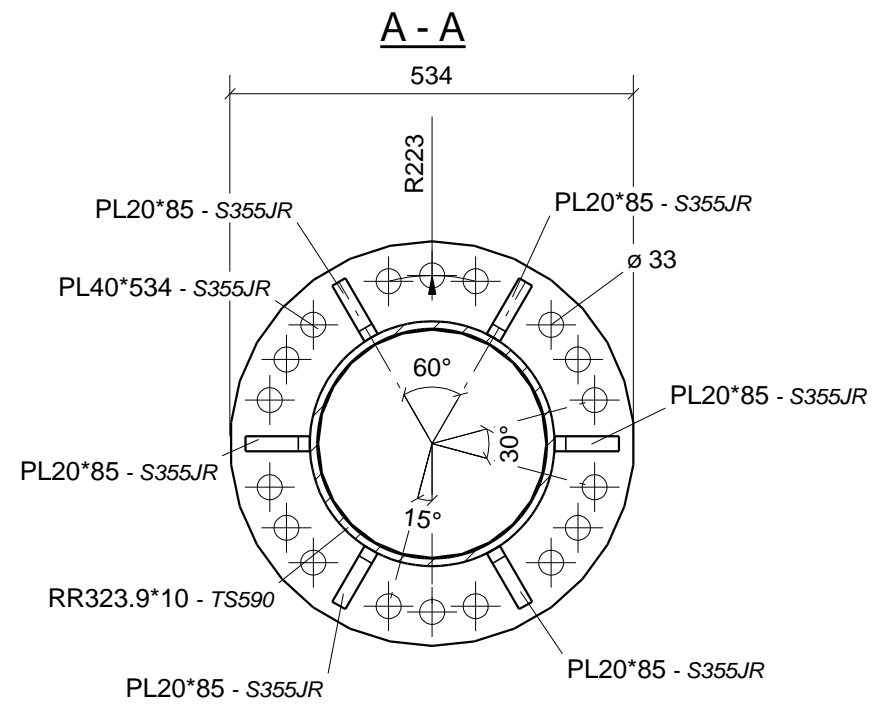
zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte :  
scostamenti ammessi per misure senza tolleranze / saldature :

**Standard Stahlbau Pichler**



Cliente: <b>UNIVERSITA TRENTO</b> Oggetto: <b>ATTEL</b>		 STAHLBAU <b>PICHLER</b> GmbH-Srl <small>STRADE 10, 39100 BOZEN/BOLZANO</small> Tel. 0471/065000 - Fax : 0471/065001 E-MAIL: info@stahlbaupichler.com	 <b>HINGE1</b>
Comessa: <b>1559</b> Reparto: <b>S</b>	Data: <b>13.11.2009</b> Scala: <b>1:10</b>		
Gez./Dis.: <b>LUKAS H.</b>	Geprüft / Controllato: <b>TB: Zelger</b>	Datum/Data: 	Geprüft / Controllato: <b>ST: Sommariva</b>

LOTTO	PEZZI
2	2

# 6 SPECIMENS



MARCA:	Descrizione:	Pezzi	Tratt.Sup.:			
<b>CSM13</b>	<b>HAUPTTRÄGER</b>	<b>6</b>				
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
e1	PL20*85	S355JR	12	140	0.0	1.3
e3	PL40*534	S355JR	2	534	0.4	47.9
e87	RR323.9*10	TS590	1	4770	4.8	369.3
Totale :					6.0	480.8

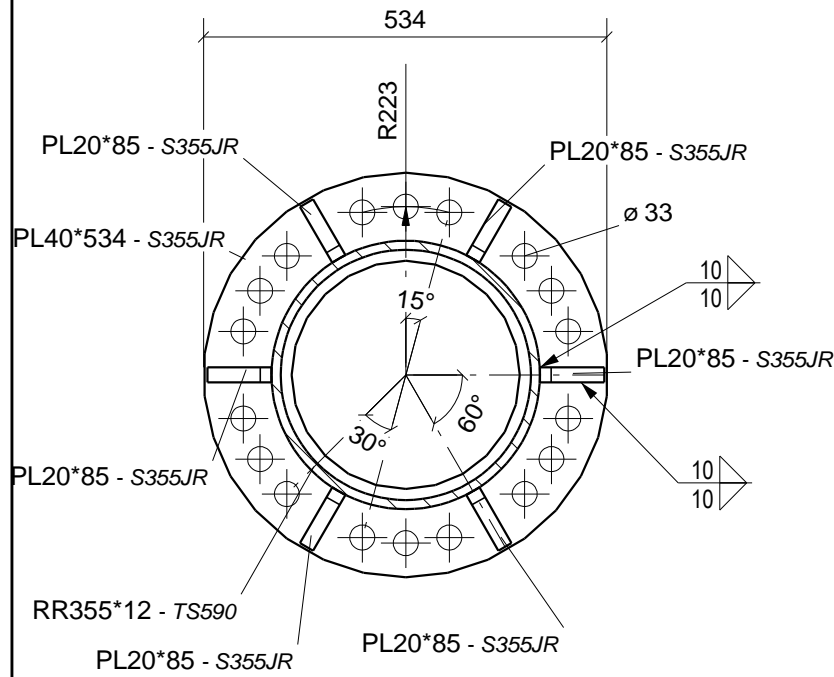
revisione secondo indicazioni del 18-10-09		LUKAS H.	11-11-2009	A
ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE		NAME - NOME	DATUM - DATA	INDEX
zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte : scostamenti ammessi per misure senza tolleranze / saldature :		Standard Stahlbau Pichler		
Cliente:	UNIVERSITA TRENTO			
Oggetto:	ATTEL			
Comessa:	1559	Data:	11.11.2009	
Reparto:		Scala:	1:10	
Gez./Dis.:		Geprüft / Controllato:	Datum/Data:	Disegno:
		TB: Zelger		<b>CSM13</b>
		Geprüft / Controllato:	Datum/Data:	
		ST: Sommariva		

LOTTO	PEZZI
2	6

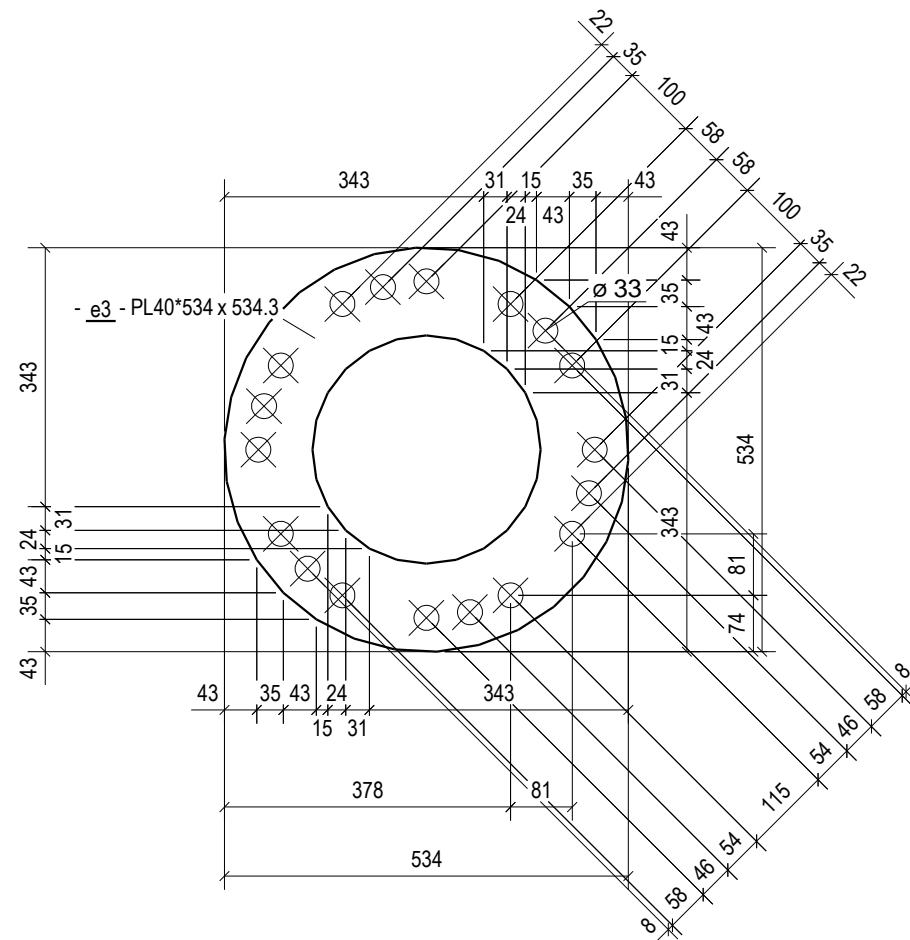
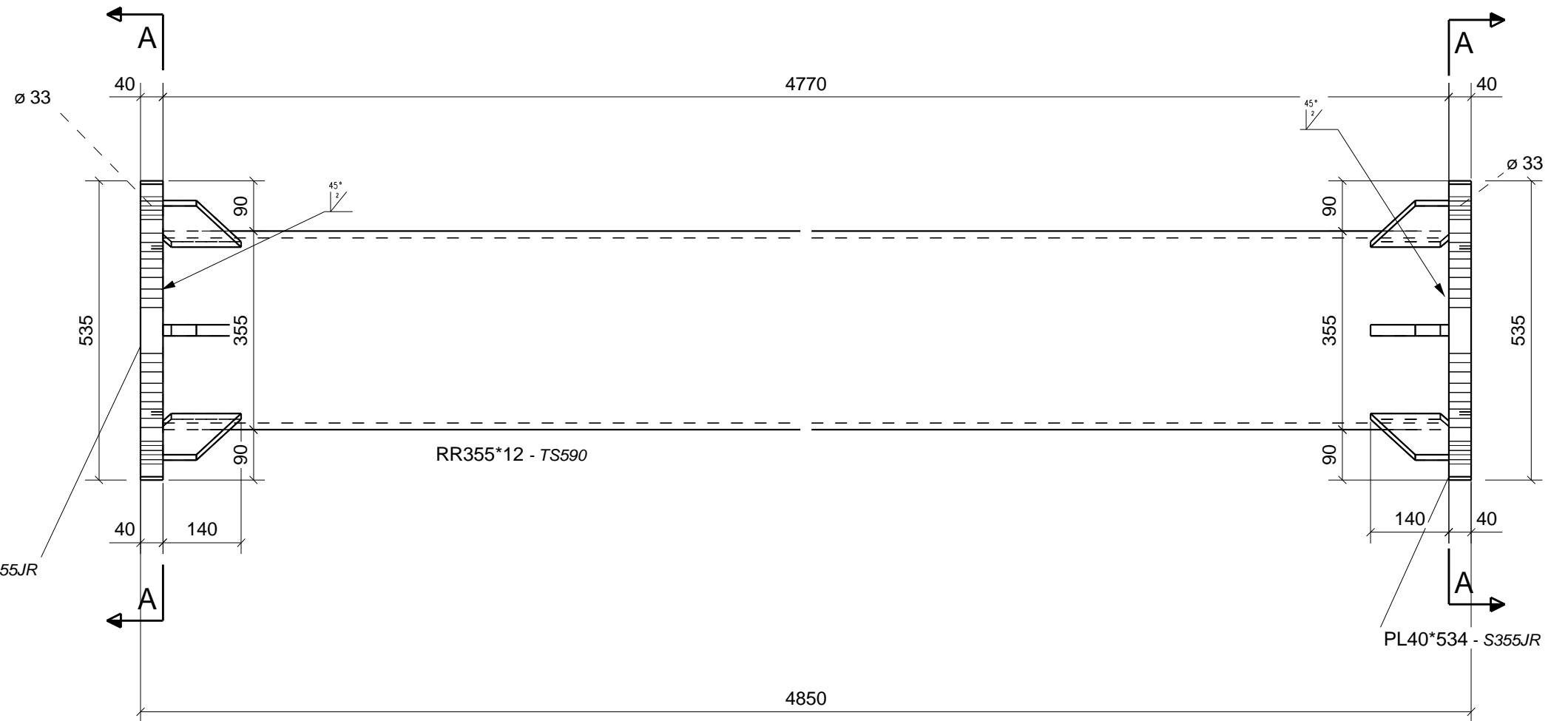


# 6 SPECIMENS

A-A



PL40\*534 - S355JR



MARCA:	Descrizione:	Pezzi	Tratt.Sup.:			
CSM11	HAUPTTRÄGER	6				
Posizione	Profilo	Mater.	Pezzi	Lungh.	Area	Peso
e1	PL20*85	S355JR	12	140	0.0	1.3
e3	PL40*534	S355JR	2	534	0.4	47.9
e6	RR355*12	TS590	1	4770	5.2	483.8
Totale :					6.3	595.3

revisione secondo indicazioni del 18-10-09		LUKAS H.	11-11-2009	A
ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE		NAME - NOME	DATUM - DATA	INDEX
zulässige Abweichungen für Maße ohne Toleranzangabe / Schweißnähte : scostamenti ammessi per misure senza tolleranze / saldature :		Standard Stahlbau Pichler		
Cliente:	UNIVERSITA TRENTO			
Oggetto:	ATEL			
Comessa:	1559	Data:	11.11.2009	
Reparto:	S	Scala:	1:10	
Gez./Dis.:	LUKAS H.	Geprüft / Controllato:	Datum/Data:	Geprüft / Controllato:
		TB: Zelger		ST: Sommariva

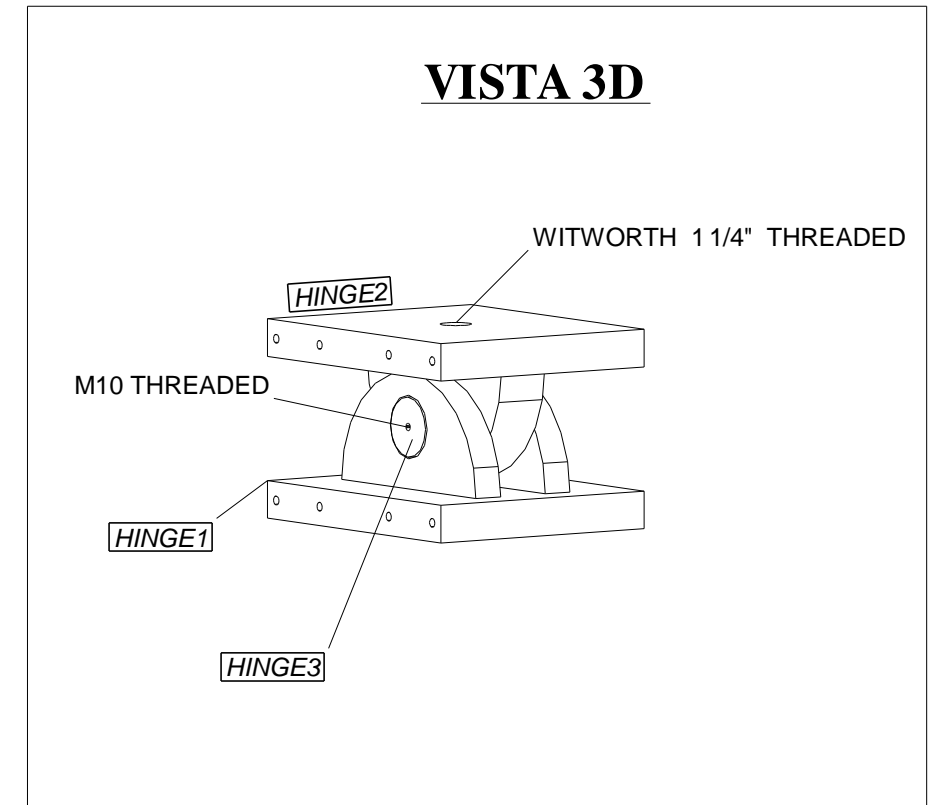
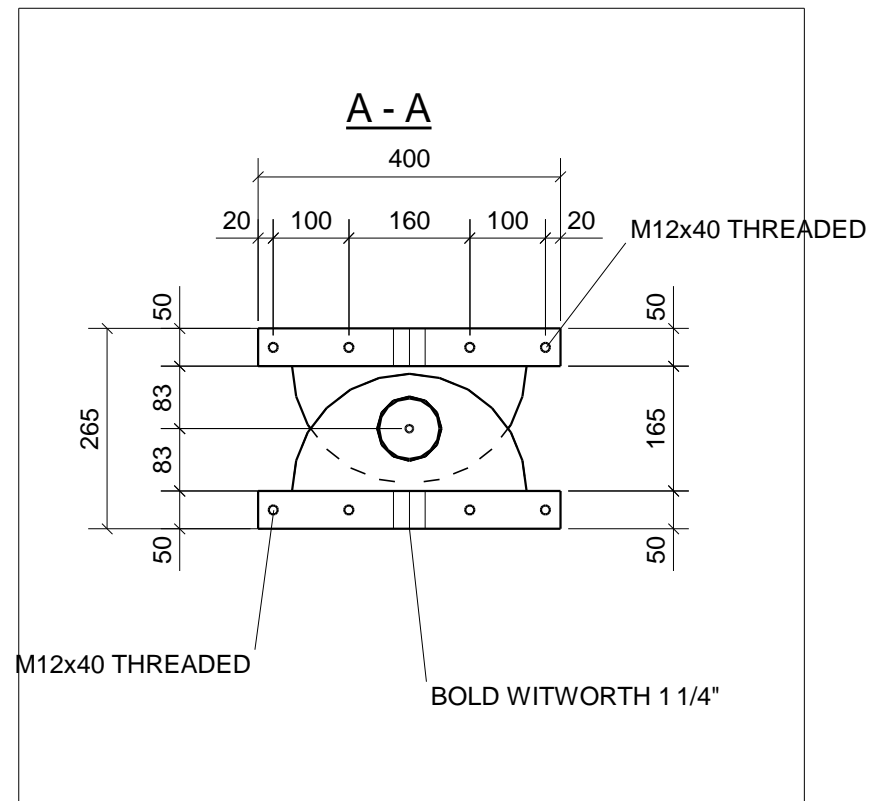
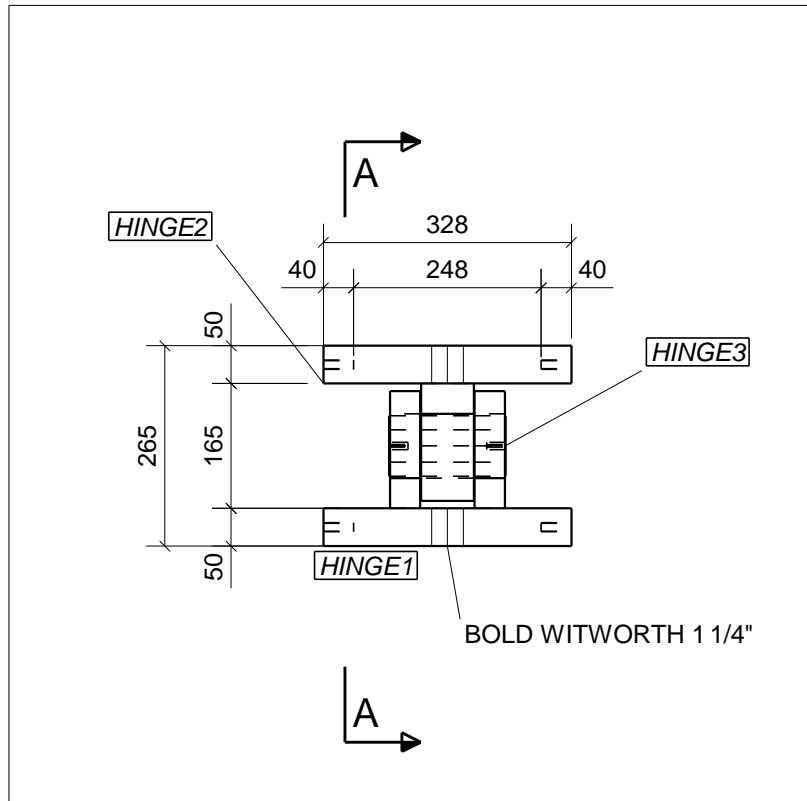

**STAHLBAU PICHLER** GmbH-Srl  
 Tel. 0471/065000 - Fax : 0471/065001  
 E-MAIL: info@stahlbaupichler.com  
 39100 BOZEN/BOLZANO


 Disegno:  
**CSM11**

LOTTO	PEZZI
2	6



# 2 SPECIMENS



FREIGABE – APPROVAZIONE

NAME/STEMPEL – NOME/TIMBRO \_\_\_\_\_

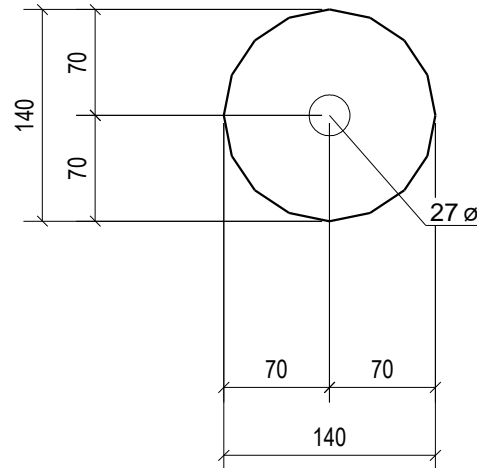
UNTERSCHRIFT – FIRMA \_\_\_\_\_

DATUM – DATA \_\_\_\_\_

revisione secondo indicazioni		LUKAS H.	28-09-2009	A
ÄNDERUNG BENENNUNG - MODIFICA DENOMINAZIONE		NAME - NOME	DATUM - DATA	INDEX
zulässige Abweichungen für Maße ohne Toleranzangabe : ENV 1090-1		Ansicht:		
scostamenti ammessi per misure senza tolleranze :		vista :		
 STAHLBAU PICHLER Tel. 0471/065000 - Fax 0471/065001 E-MAIL: info@sta-bspichler.com 39100 BOZEN/BOLZANO - VIA EDISON-STRASSE 15		FREIGABE / APPROVATO		
		ABTEILUNG / REPARTO	DATUM / DATA	UNTERSCHR. / FIRMA
		TB		Zelger
		ST		Sommariva
BAUHERR / COMMITTENTE		UNIVERSITA TRENTO		
BAUSTELLE / CANTIERE		MASSSTAB / SCALA		
BENENNUNG / DESCRIZIONE		INDEX / REV.		
ATTEL		A		
ATTEL				
HINGE		Z.Nr.-ds.n.		
		1559-S-P110		


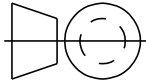


# 32 SPECIMEN



LOTTO	PEZZI
2	32

Quantità	per Marca
32	UNITN25

Cliente: UNIVERSITA TRENTO	Comessa: 1559	Reparto: S	Disegno: PLATTE	 <b>STAHLBAU PICHLER</b> GmbH-Srl Tel.0471/065000 - Fax : 0471/065001 E-MAIL: info@stahlbaupichler.com 39100 BOZEN/BOLZANO	
Descrizione: PLATTE	Profilo: PL25*140	Quantità: 32	Peso: 2.9		
Lunghezza: 140	Posizione: UNITN25	Data: 24.09.2009	Modifica:		
Superficie: 0.04			Rev:		
Materiale: S355JR					