POLITECNICO DI TORINO

Master’s Degree in Civil Engineering

Master’s Thesis

Precast elements in the construction site

Analysis of the use of Composite Steel Trussed Concrete Beams (CSTCBs) in a building in Cirié (TO)

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1. <u>INTRODUCTION</u>

1.1 INTRODUCTION TO THE STUDY AND MAIN OBJECTIVES

The construction industry has undergone a deep change in the last decades and significantly turned into industrialized processes for better quality, productivity and sustainability. Precast technology is one of these innovations which provides some advantages including shorter construction time, better quality and safer jobsite.

Precast elements are manufactured in prefabrication factories and assembled on-site and are usually installed more quickly and with more predictable costs.

In this context, the Composite Steel Trussed Concrete Beams (CSTCBs) represent a modern structural solution, integrating the best features of steel and concrete in precast systems. CSTCBs are composed of a steel truss to serve as permanent formwork and tensile reinforcement, combined with cast-in-place concrete. This hybrid characteristic makes the beams suitable for both temporary and permanent construction stages. In the initial construction stage, the steel truss supports the load while concrete is set (Phase I); once concrete hardens (Phase II), the composite system becomes continuous and achieves full structural capacity.

The application of CSTCBs is particularly interesting in medium-and large-size buildings, where structural effectiveness and rapidity of construction are the most important aspects. Their use is consistent with environmental cost and modularity requirements in present day.

The current thesis studies the application of CSTCBs in a real construction project, located in Ciriè (TO) Italy, including the analysis of the technical development and the practical impact of the adoption of this solution on the construction site. Furthermore, a comparison with traditional RC beam solution is presented in terms of structural performance and design efficiency.

Finally, the goal is to critically analyse the advantages and drawbacks of CSTCBs as well as to explore the possibilities for its further development in contemporary construction.

1.2 STRUCTURE OF THE THESIS

The thesis is structured into five main chapters, each one analysing a specific issue of the research.

Chapter 1 (current section) is a brief introduction to the study, useful to understand the context and the main objectives.

Chapter 2 is an extensive overview of precast elements, its historical aspects, types and materials as well as the methods of structural analysis. It also provides a review of present design methodologies, connection techniques, and the applicable standardization framework.

Chapter 3 focuses specifically on precast beams, with specific care to Composite Steel Trussed Concrete Beams (CSTCBs). This section introduces the typologies and classification of CSTCBs, followed by an in-depth analysis of their structural behaviour, including design regulations, phased performance and time-dependent effects such as creep.

Chapter 4 presents the case study of a real building in Ciriè (TO), where CSTCBs have been implemented. The section includes a general description of the building and its structural model, a discussion of design choices, and a comparison between CSTCBs and traditional RC beams. The application of CSTCBs in the construction site is also examined, highlighting practical aspects such as transport, assembly and logistics.

Chapter 5 summarizes the main findings of the study and provides conclusions and suggestions for future research and development, with the aim of promoting broader use of CSTCBs in structural design.

2. PRECAST ELEMENTS

2.1 GENERAL CHARACTERISTICS OF PRECAST ELEMENTS

When dealing with precast concrete elements a lot of definitions can be achieved from different references:

- "Precast concrete elements are structural components such as beams, columns, walls, or floor slabs that are cast and cured in a controlled manufacturing environment and then transported to the construction site for installation." [1]
- "Precast concrete elements refer to individual units, such as pipes, manholes, panels, or bridge segments, that are produced in a plant and designed for specific structural or architectural functions in construction projects." [2]
- "Precast concrete elements are components that are cast in a reusable mould off-site and cured in a controlled environment, then transported to the site for installation. These include structural elements like floor slabs, stairs, and wall panels." [3]

So, it's possible to notice that precast concrete elements are structural or architectural components manufactured by casting concrete in reusable moulds under carefully controlled environmental conditions, typically at an off-site facility before being transported to the construction site for assembling process. These components can include beams, columns, wall panels, floor slabs, stairs, and various utility structures, tailored to their specific functional requirements. The Precast/Prestressed Concrete Institute (PCI) [1] states that these elements are designed to meet stringent performance and quality standards, providing benefits such as consistency, durability, and quicker installation times. Similarly, the National Precast Concrete Association (NPCA) [2] highlights the versatility of precast concrete, emphasizing its wide-ranging applications in infrastructure, commercial, and residential projects. Additionally, the British Precast Concrete Federation [3] underscores how these elements support modular construction methods, promoting efficient production processes while reducing on-site labour demands.

In general, what really distinguish precast concrete from cast in situ is its stress and strain response to external and internal effects. The external effects are represented by the load applied on these elements while the internal ones include autogenous volumetric changes, these are grouped in the section of "actions" in the Eurocodes, specifically for precast structures the main references are the basic code EC0 (BS EN 1990 2002), the loading or actions code EC1 (BS EN 1991-1-1 2002) and the concrete design code EC2 (BS EN 1992-1-1 2004). An example of an architectural-structural precast concrete frame is shown in Figure 1.

Figure 1 Architectural-structural precast concrete frame at Scottish Office, Leith, United Kingdom.

Kim S. Eliott in his book [4] empathises some specific factors that distinguish precast concrete from other forms of construction, particularly regarding the performance and design requirements for connections and load transfer mechanisms. By definition, a precast concrete element has fixed dimensions and must be connected to adjacent components to create a complete structural system. In simpler applications, this connection can be made using features like bearing ledges or corbels, as demonstrated in Figure 2. However, in case of volumetric changes induced by thermal shrinkage or load-induced strains and often manifesting as shortening and lengthening, differential movement between precast elements may develop (Figure 3a). The friction at the contact surfaces resists such movement, generating a force expressed as $F = \mu R$, where μ represents the coefficient of friction and R denotes the normal force. This frictional resistance may induce splitting forces within the components, potentially leading to failure if the necessary reinforcement is not in place (Figure 3b). A practical example of this issue is shown in Figure 5a, where relative movement between unreinforced precast slabs and their supporting beams caused spalling in the beam (process of breaking off of flakes from the concrete surface). In other cases, restricting relative displacement or end rotation has led to unintended positive bending moments, resulting in cracking on the bottom side of slabs or at beam-to-column corbel connections (Figure 5b).

Figure 2 Simple bearing nib and corbel.

Additionally, flexural rotation of suspended members, such as beams, may reduce the effective bearing length (l_b), which increases stress levels at the bearing interface. This stress concentration could cause localized crushing at the top of the supporting column unless a bearing pad is used to evenly distribute the load (Figure 3c). In scenarios where the bearing surface is narrow, stress dispersion from the interior toward the exterior of the column can create lateral tensile strains, which might lead to splitting or bursting of the concrete beneath the bearing zone. Properly designed reinforcement details can help mitigate this form of failure (Figure 3d).

If the column is subjected to an accidental or structural force, H, such that H exceeds μR, the resulting displacement u, will not be elastically recoverable. This may lead to instability or a complete loss of bearing unless the bearing possesses adequate shear capacity (refer to Figure 4e). Furthermore, failure of the column's foundation will result in a loss of bearing unless the bearing is specifically designed to provide sufficient tensile capacity (refer to Figure 4f).

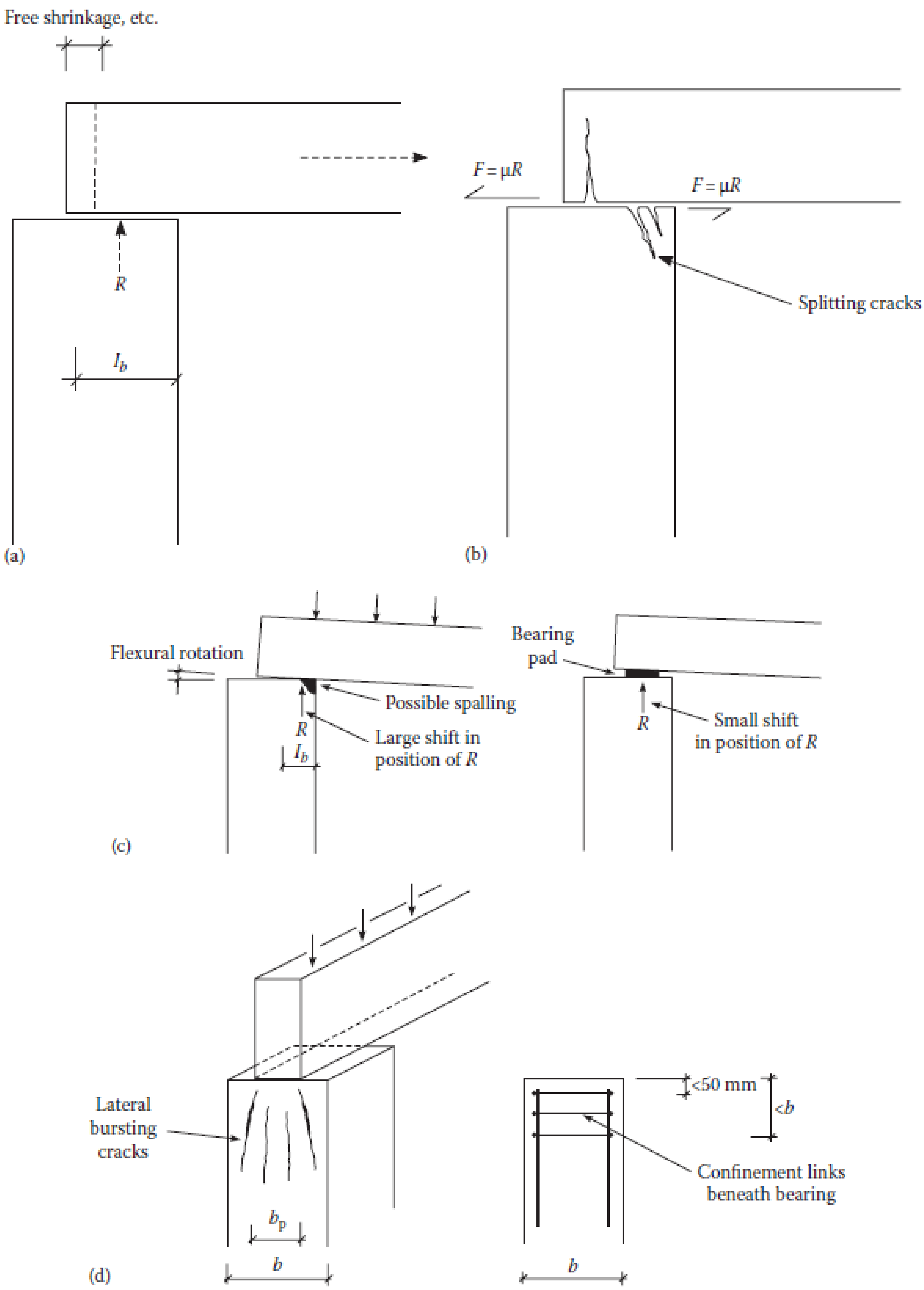

Figure 3 (a) Unrestrained movement between two precast concrete elements. (b) Restrained movement but without tensile cracking prevention. (c) Reduced bearing length and stress concentrations due to flexural rotation. (d) Lateral splitting due to narrow bearings (left) and confinement rebars (right).

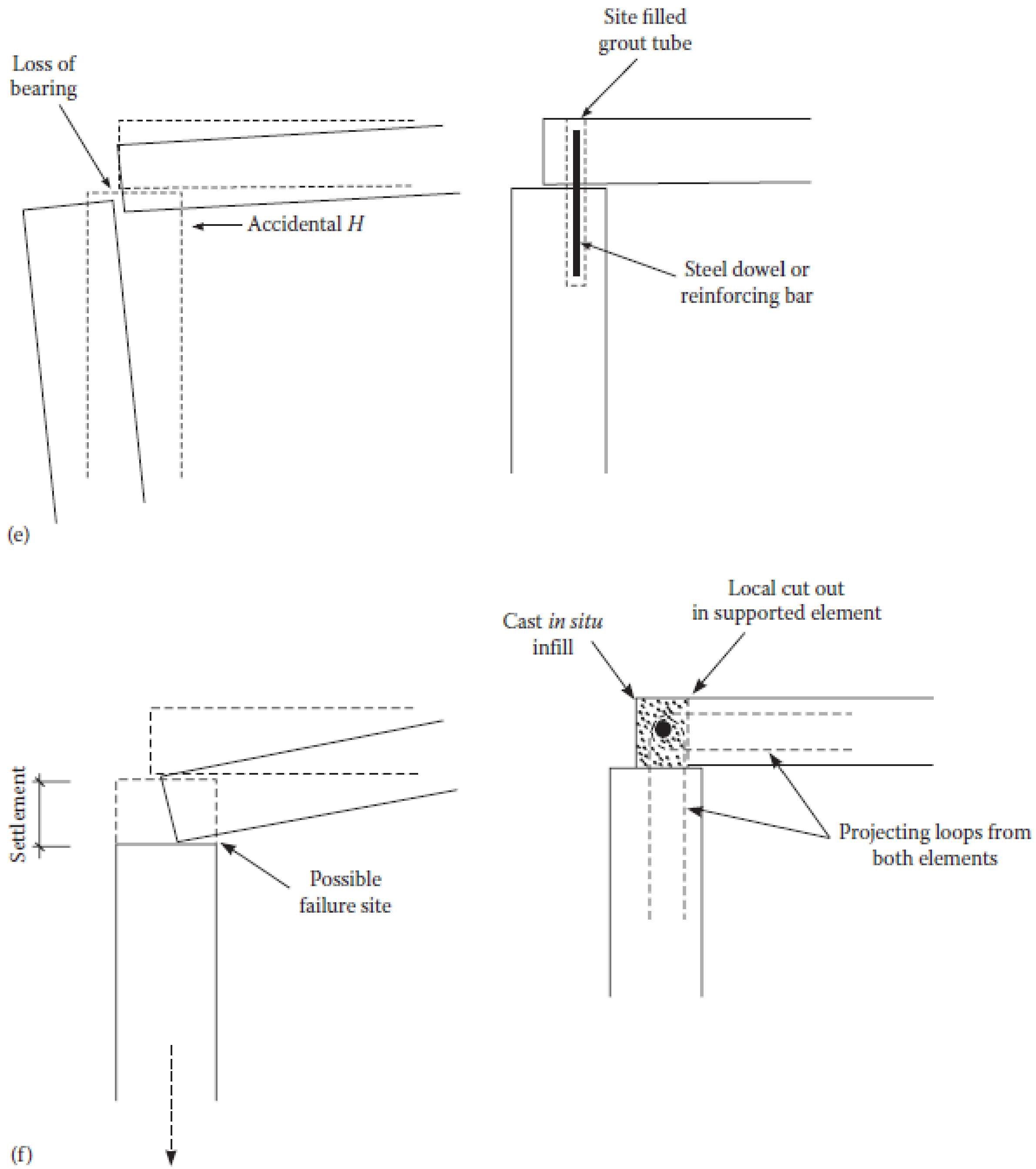

Figure 4 (e) Loss of bearing due to accidental actions (left) and preventative dowel bar (right). (f) Loss of bearing due to settlement (left) and rebar loops (right).

(a) (b)

Figure 5 a) Spalling due to relative movement between slabs and beams. (b) Cracking due to spurious restraint at a beam-to-column corbel connection.

The assemblage of different types of precast elements leads to the formation of a precast concrete structure, a three-dimensional framework able to resist gravitation, wind and earthquake loads. The best application of these structures are ideally buildings requiring minimum internal obstruction such as offices, car parks and schools.

In a generic frame the structural elements that bear the loads are columns, beams, floor slabs, staircases and stair cores. From Figure 6 it's evident how the quantity of concrete in a precast frame is less than 4% of the gross volume of the entire building and two-thirds of this is in the floors; specifically in the middle plane of the longest side of the building can be seen that it is braced against horizontal actions and in the shortest is present a diagonal bracing. In this typology of frame the connections are idealised as hinges and so the bracings are needed to provide lateral stability in both directions.

Other solutions are those of Figure 7 and Figure 8. In both cases the frame is unbraced, but in the first the resistance against horizontal actions is provided by the same columns that support vertical actions, while in the second is represented a moment resisting frame where the resistance against horizontal actions is given by the entire frame action thanks to the moment resisting beam to column connections (really demanding and rigid connections able to transfer the bending moment).

A defining characteristic of the precast framework is that beam-to-column connections are typically not fully rigid, often referred to as "semi-rigid." As a result, the columns must also withstand horizontal forces, similar to the behaviour of an unbraced frame illustrated in Figure 7.

There are also more complex solutions made by the combination of the previously described such as:

- Partially braced structures which are a blend of unbraced, above or below a certain level, and braced structures;

- Unidirectionally braced structures which are braced only in one direction and unbraced in the other.

Figure 6 Precast concrete skeletal braced frame, The "Green Apple" retail centre and car park, Helsinki.

Figure 7 Precast concrete "skeletal" sway frame, Europark, Rome.

Figure 8 Precast concrete "skeletal" known as "semi-rigid" frame, Recife University, Brazil.

The growth of the precast concrete industry, combined with the successful execution of numerous construction projects using precast components, highlights the practicality and cost efficiency of this construction approach. Globally, precast structural frames without architectural finishes, commonly referred to as "grey" precast, represent about 5% of the multistorey building market. Meanwhile, precast systems that include integrated façades or decorative elements account for a considerably larger global market share of approximately 15%. In regions with colder climates or where on-site labour costs are exceptionally high, this share can rise to as much as 70%.

Despite that it is widely believed that in certain countries precast cannot compete economically with cast in situ concrete, especially where the labour-to-material cost ratios are low. In areas where labour regulations emphasize the extensive use of unskilled site workers, incorporating heavy precast concrete components may raise specific safety challenges concerning transportation, handling, and temporary stability during installation.

Additionally, in countries with a well-developed steel industry and significant expertise in steel design, there is often a perception that precast concrete cannot economically compete with structural steel framing systems. Even so, precast concrete frequently emerges as the most practical and cost-efficient choice under certain conditions. This is particularly true for projects requiring accelerated construction schedules and improved quality control.

Kim S. Eliott [4] highlights how the key to achieving success with any type of mixed or precast concrete solution lies in providing the client, architect, and consulting engineer with a solution that is:

1. Buildable

   a) The construction sequence is sensible enabling other trades to merge into programmes.
   b) The construction is safe and temporary stability is guaranteed.

c) Economic cranes are used.
d) Labour is skilled.

2. Cost-effective

a) Building components cost-per-structural capacity is comparable to that of other materials, for example cost/bending moment or cost/axial load capacity.
b) The overall building costs, inclusive of transport, fixing finishing, maintenance and repair, are competitive.
c) Production is quality-assured.
d) The solution uses factory-engineered concrete.
e) Destructive load testing or non-destructive testing assurances are given.

3. Fast to erect

a) Although the precast manufacturing period may be several weeks, once started, construction proceeds rapidly.
b) Following trades (bricklayers, electricians, joiners) move in quickly.
c) Handover or possession can be phased.
d) Clear heights and floor zones are satisfied.
e) Service routes are not interrupted.
f) Beam, column sizes and bracing positions are satisfied.
g) Decorative concrete, both internally and externally, may be exploited.

Another important factor is that prefabrication factories are not cheap to run. In certain cases, it may be more cost-effective to set up a prefabrication facility directly on-site or in a nearby field and operate from there. Regardless of the chosen approach, the factory must be capable of performing tasks that cannot be accomplished at the construction site.

In general, is difficult to rate or weight the various advantages that precast concrete has over in situ construction. Each application has to be considered on its merits in specific circumstances. Advantages inherent in precasting are the introduction of factory disciplines, increased accuracy and improved standards of finish, reduction in site labour requirements, the removal of large sections of work from the critical path determining overall contract duration.

A deep analysis of the advantages and drawbacks considering the specific types of construction will be provided in the next chapters.

2.2 HISTORICAL NOTES AND DEVELOPMENT

Historically, the prefabrication of concrete and the development of precast concrete structures for residential, commercial, and industrial buildings have met four major phases:

1. The Developing Years (1920 - 1940): This period saw major technological advancements, including the introduction of prestressed concrete (PSC) and significant improvements in reinforced concrete (RC). These innovations led to stronger materials, optimized structural designs, and increased durability and resilience of precast elements.
2. The Mass Production and Standardisation Period (1945 - 1970): In the post-war era, the reconstruction of European housing and the development of South-east Asia were largely driven by standardized, mass-produced precast systems. Wall panel construction became prevalent and semi-automated production of floor slabs, such as prestressed long-line extruded or slip-formed hollow core units (HCUs), faced big improvements.
3. The Lightweight and Long-Span Period (1970 - 2000): Driven by the demand for more efficient structural systems, this era emphasized lighter construction with greater span-to-depth ratios. This was achieved through composite, continuous, and integrated designs, using hybrid systems (combining precast and in-situ concrete) and mixed materials such as steel, timber, and masonry.
4. The Thermal Mass Period (2000 - present): In response to growing sustainability concerns, this phase emphasizes the environmental benefits of factory-engineered concrete and off-site construction methods. Innovations focus on enhancing energy storage, increasing thermal admittance, and reducing heat transmittance (U-values).

   In reality there’s now a new era which is yet established in some countries increasing the tendency for automated manufacture and off-site prefabrication:

5. The Automated Period involving the so called “MMI”. “MMI” stands for modernisation, mechanisation and industrialisation for the design, detailing and manufacture of concrete structures. This phase is further enhanced by the integration of Building Information Management (BIM), enabling coordinated control over building services, structural elements, scheduling, and construction.

Each phase will be covered in detail in the following paragraphs, meanwhile Table 1 gives an historical summary of the main developments in the precast industry.

Period	Materials	Technology	Floors	Skeletal frames	Wall frames	Facades
Developing 1920–1940	Higher strength C50.	Ferro cement products. Reinforced. Prestressed	Ribbed slabs. X-beams and blocks. Wet cast hollow core.	Limited height, small regular spans, some psc X-beam and block.	Limited height with insitu or X-beam floors.	Precast panels.
Mass production 1950–1970	Calcium chlorides. HAC. Low-slump. RHPC White OPC	No-fines concrete. Sandwich panels. Steel and polypro fibres, thin wall formwork. Half-slab, semi-automated cages.	Solid or ribbed slabs. Double-tee and single-tee. Extruded & slip-formed hcu (≤250)	Standardised. Modular grids. National Building Frames. Patented connections systems. Steel rafters and Mansard roofs	Wall and cross-wall frames, 12–25 storeys. Mass production. Bathroom pods. Ronan Point collapse.	Exposed aggregates. Sandwich panels. Load bearing wall panels.
Lightweight and long-span 1970–2000	Lightweight aggs. Colouring pigments. SCC. Steel fibre high bond. HSC to 100 N/mm^2. Reactive powder up to 250 N/mm^2.	Stability ties for progressive collapse. Brick slips. Composite and continuous design. Semi-rigid connections. Generic algorithms for production.	Prestress beam and polystyrene blocks. 400 mm deep hcu. Deeper hcu for specials (730). Diaphragm action tests. Bubble-deck. Carousel casting plant. Cast-in lifting hooks in hcu.	Hybrid frames. Integrated frames. Longer composite spans. Mixed construction. Semi-rigid and PRESSS research, NIST connector.	Knee-joint open frames (wall-hcu). Single-span twin wall. Split skin.	Coloured, painted. Polished. Blast resistance. Large area panels.
Thermal and acoustic 2000–2010	Blended cements. PFA and GGBS. HSC 120 N/mm^2. Rapid hardening SCC	Thermal and acoustic design.	500 deep hcu. Thermally insulated ground floor beam-and-block floors and floor slabs	22 storey seismic frame (NIST joints)	Twin-wall (precast-insitu infill). Up to 54 storey with insitu cores (Netherlands)	Thermal mass and insulated.
MMI in the future						

HSC = High-strength concrete, HAC = high alumina cement, OPC = ordinary Portland cement, RHPC = rapid hardening Portland cement, PFA pulversized fuel ash, GGBS = ground granulated blastfurnace slag, SCC = self compacting concrete, hcu = hollow core floor unit.

Table 1 Historical developments in precast concrete technology.

Over the course of these periods, the height of multi-storey buildings has steadily increased, driven by advances in technology, evolving architectural demands in various countries and cities, and notable improvements in the compressive strength of concrete [5]. As shown in Figure 9, since 1950, the compressive cube strength of concrete, and correspondingly, the axial load capacity of a 300 × 300 mm reinforced concrete (RC) column, has increased threefold, from approximately 2000 kN to nearly 6000 kN. This improvement is also attributed to the enhanced yield strength of high-tensile reinforcement, which has risen from 410 to 500 N/mm². For comparison, cold-worked mild steel with a yield strength of 250 N/mm² was commonly used in precast columns up until the 1960s.

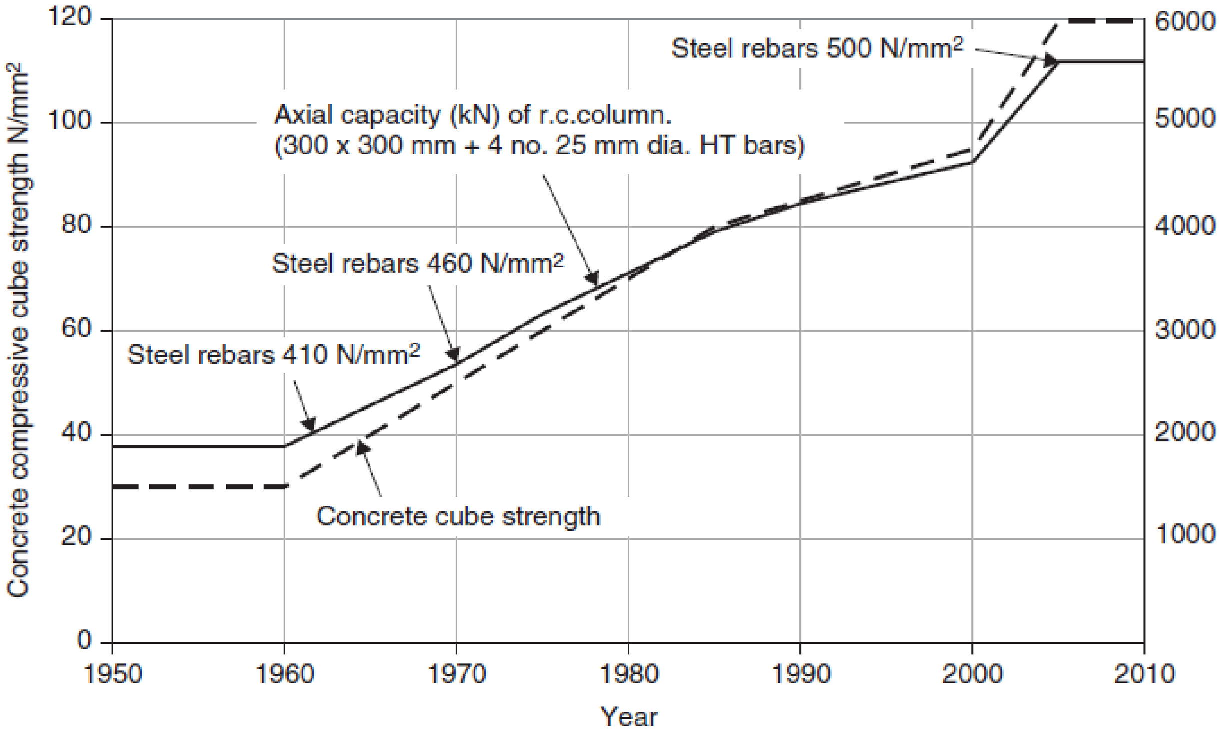

Figure 9 Approximate compressive strength (left ordinate) used in the precast industry since 1950s, and the corresponding axial load capacity of a 300 × 300mm reinforced concrete column (right ordinate).

This significant increase in material strength translates into a reduction in column size for the same axial load. For instance, to support an ultimate axial load of 2000 kN in 2010, a column section would need to be only 145 mm wide, down from 300 mm, representing a 48% size reduction. While slenderness effects can offset these gains to some extent, the overall trend remains evident. Moreover, updates to structural design standards, such as the introduction of Eurocode EN 1991, which reduced ultimate load demands by approximately 10%, have further reinforced this trend.

As a direct consequence, building heights have increased. Figure 10 illustrates the total building height, or number of storeys, achievable using either fully precast frames (dashed line) or skeletal/wall frames braced with in-situ concrete cores. The rise in skeletal frame construction closely follows the improvements in concrete strength and column capacity shown in Figure 9, highlighting these factors as key drivers in the evolution of tall precast buildings. A noticeable leap in building heights post-2000 corresponds with the development of readily achievable high-strength, high-performance concretes innovations emerging from concrete research laboratories in

Scandinavia, France, and the USA. The rises and falls of wall frames have been associated mainly to social trends.

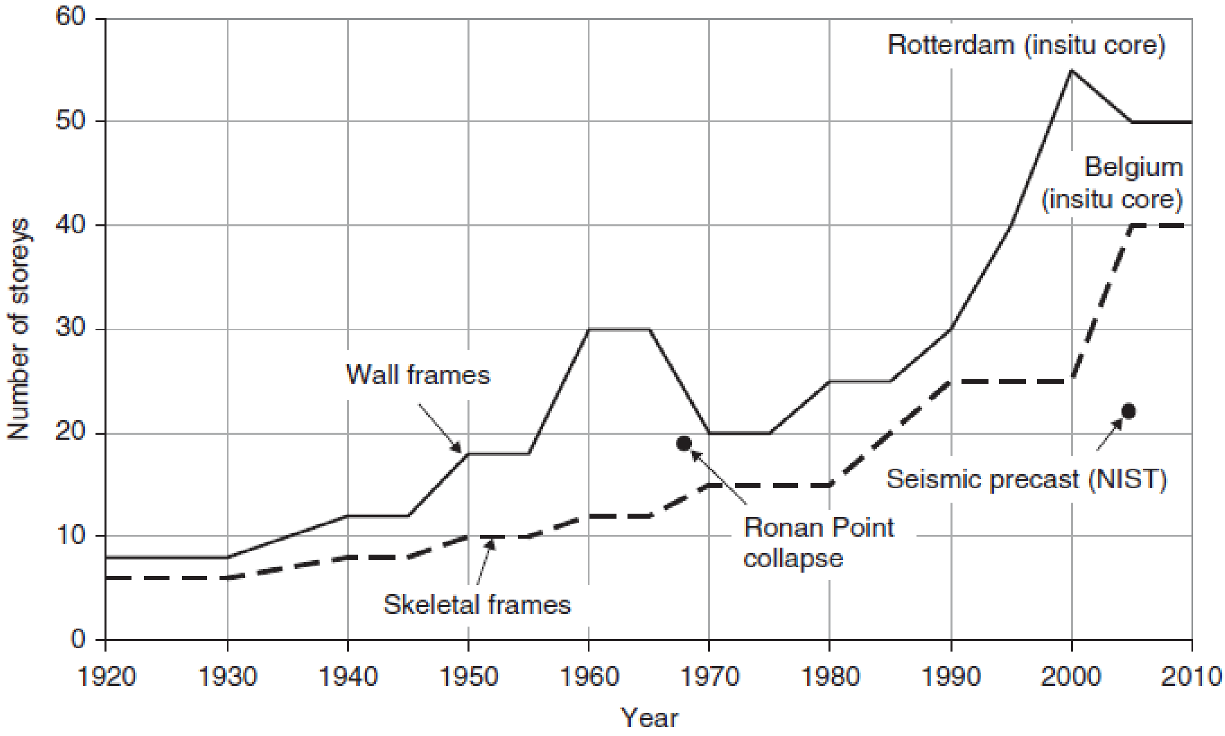

Figure 10 Typical and approximate maximum heights for precast concrete skeletal and wall frames since 1920s.

In general, advancements in concrete strength, along with increased capacity of columns, beams, and slabs combined with reductions in structural zones and column dimensions, enabled engineers to refine and optimize the design of precast reinforced concrete (RC) and prestressed concrete (PSC) frames. These developments are the basis of the Lightweight and Long-Span Period, spanning approximately from 1970 to 2000.

2.2.1 The Developing Years (1920 – 1940)

A brief summary of the Developing years was provided by [6]:

"The reason why precast concrete came into being in the first place varies from country to country. One of the main reasons was that availability of structural timber became more limited. Some countries, notably the Soviet Union, Scandinavia and Northern Continental Europe, who together possess more than one-third of the world's timber resources but experience long and cold winters, regarded its development as a major part of their indigenous national economy. Structural steelwork was not a major competitor at the time outside of the United States, since it was batch processed and thus relatively more expensive. During the next 25 years developments in precast frame systems, prestressed concrete long span rafters (up to 70 feet), and precast cladding, increased the precasters' market share to around 15 per cent in the industrial, commercial and domestic sectors. Influential articles in such journals as the "Engineering News Record" encouraged some companies to begin producing prestressed floor slabs, and in order to provide a comprehensive service by which to market the floors these companies diversified into frames. In 1960 the number of precast companies manufacturing major structural components in Britain was around thirty. Today it is about eight."

This period can be easily subdivided into two sub-periods:

1) Initially these structural systems were not planned with the need to minimise structural zones, which were often around 800 - 1000mm in offices, giving rise to span-depth ratios of less than 10. This was called "heavy period".
2) Approaching 1970's the cost of structural components began to increase and engineers were forced by architects to reduce the structural zones. In this way the "lighter period" developed and improved the structural efficiency as shown in Figure 11.

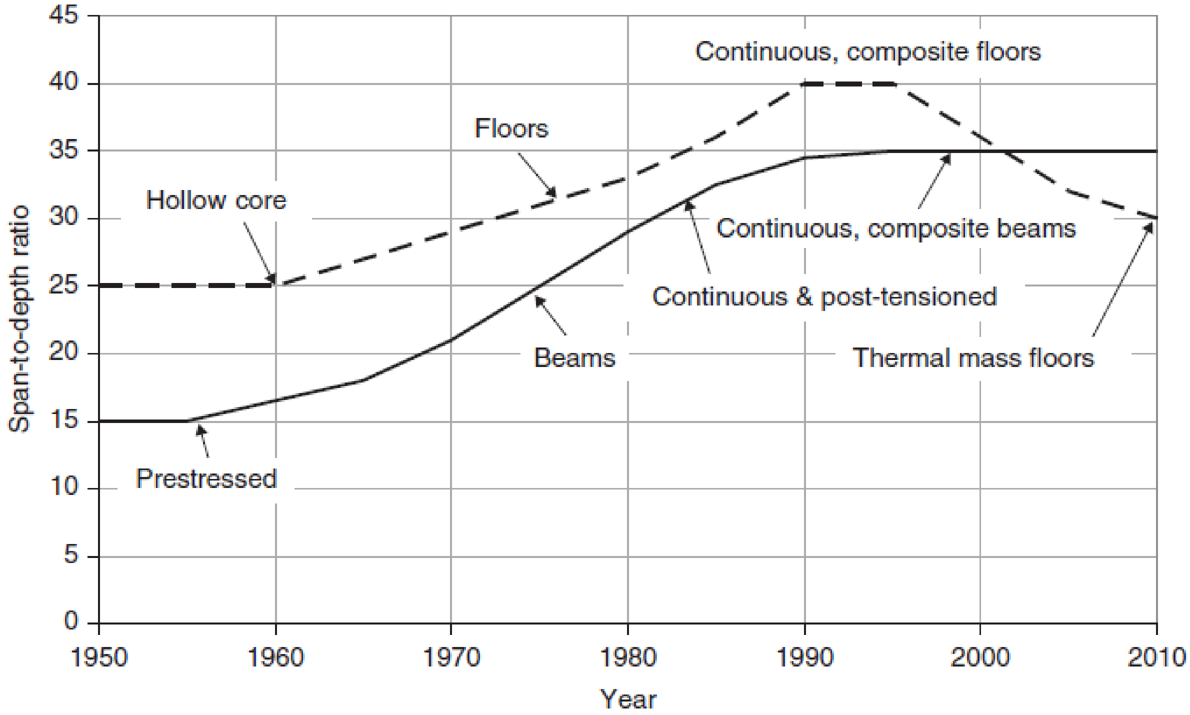

Figure 11 Approximate span-to-depth ratios with some key designs for precast concrete floors and beams since 1950s.

2.2.2 The Mass Production and Standardisation Period (1945 – 1970)

During this period the designers developed the concept of standardisation through the publication of a standardised structural system or framework used nationally for building with precast components called NBF (National Building Frames) and a standardised design known as PBF (Public Building Frame). These structural models referred mainly in the use of a fixed framework (using columns, beams, slabs and wall panels) that is modular, repeatable and standardised in order to unify the construction processes across projects or regions. An example of the application of these models into a real structure is shown in Figure 12 and Figure 13.

In the following years prefabrication evolved toward a more client-based concept with a more creative use of precast components leading to a new generation of precast concrete frame near to those of these days.

Figure 12 National Building Frame in UK. Highbury Technical College (now Highbury College) Portsmouth, opened in September 1963.

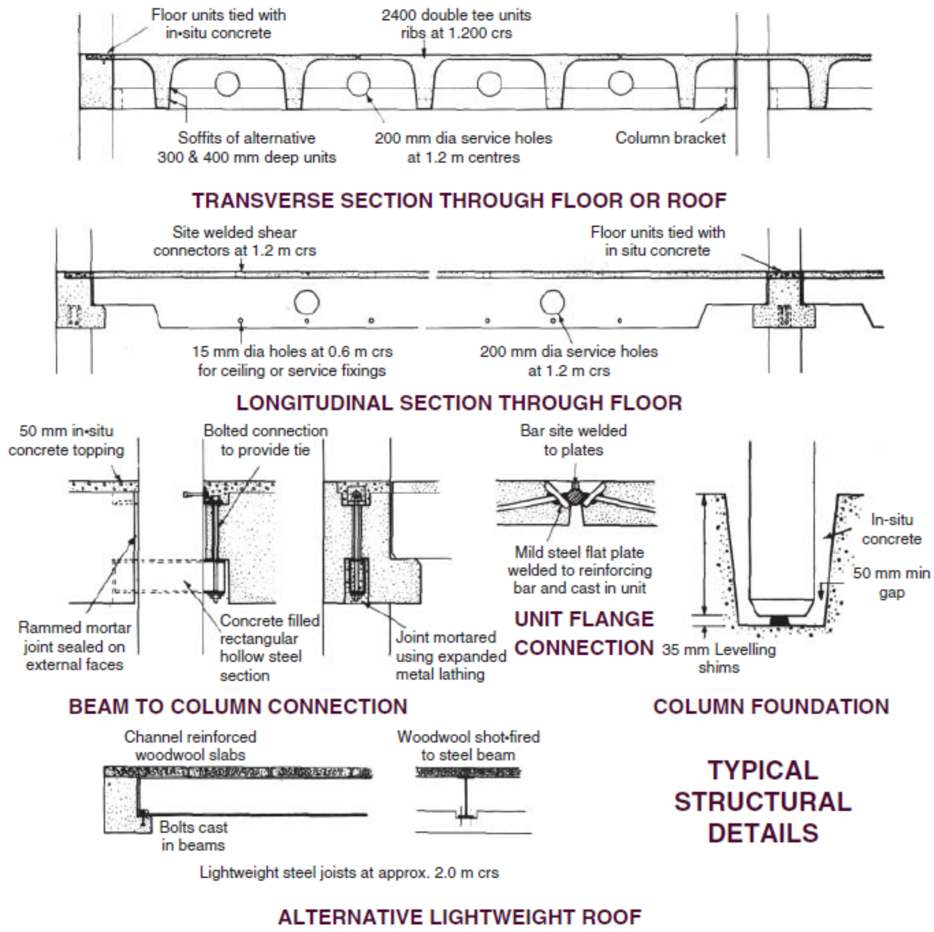

Figure 13 National Building Frame components.

The development of precast elements in buildings followed different trends among the world, Nilson [7] made a comparison between the European and American situation: "Over the past 30 years, developments of prestressed concrete in Europe and in the United States have taken place along quite different lines. In Europe, where the ratio of labour cost to material cost has been relatively low, innovative one-of-a-kind projects were economically feasible. In the U.S. the demand for skilled on-site building labour often exceeded the supply, economic conditions favoured the greatest possible standardisation of construction…"

Very often there's still a misunderstanding between the concepts of "modularisation" and "standardisation". Industrial modularized buildings were introduced in the 1950s in the heavy construction period following the Second World War and are characterised by zero flexibility off the modular grid. On the other hand, Standardisation refers to the use of a predefined set of components that can be flexibly assembled in various ways; by varying beam depths, column lengths, wall placements, and other parameters, these same components could be rearranged to create entirely different structures. In contrast, such adaptability is not achievable with modular systems, which rely on fixed, repetitive units.

2.2.3 The Lightweight and Long Span Period (1970 – 2000)

The years 1970 - 1990 saw a major shift in the design of precast buildings leading to an optimisation of the floor and beam components by reducing the use of materials and creating lighter and more structurally demanding structures.

This was possible due to important improvements in the manufacturing techniques and due to a series of key factors such as:

- An increase in concrete compressive strength from C35 to C50 (f_{ck} = 35 to 50 N/mm²)
- The introduction of cement additives and admixtures, including air-entraining agents;
- Higher levels of prestress applied to strands and wires, reaching up to 75% of their ultimate strength;
- An increase in the ultimate tensile strength of tendons, from approximately 1600 to 1860 N/mm².

Collectively, these innovations resulted in precast units that were 10 - 15% lighter and offering 25 - 35% greater bending capacity.

However, increasing spans and reducing the mass means also higher deflections and lower natural frequency. Figure 14 illustrates how changes in span, depth, and self-weight influence the natural frequency (*f*), specifically, the first fundamental mode of a non-composite prestressed hollow core unit (HCU), assuming a dynamic damping factor of 0.05 to account for the presence of partitions and walls.

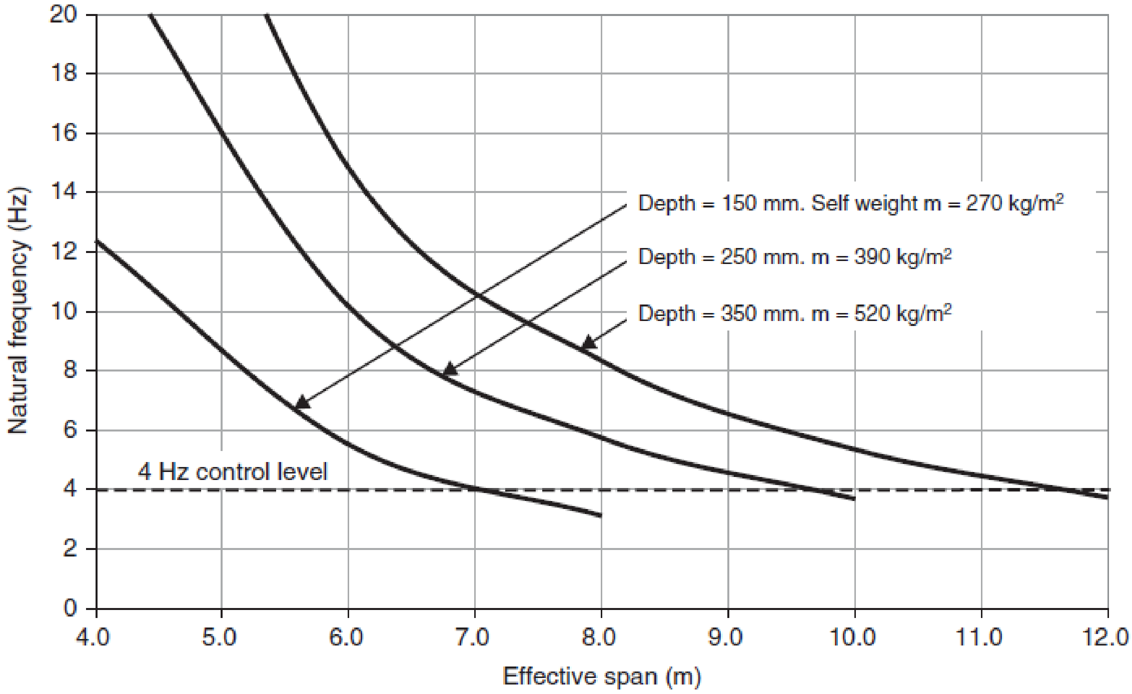

Figure 14 Relationships between natural frequency, floor mass and span for prestressed hollow core floor units.

The data reveal that a 33% reduction in self-weight or surface mass (from 520 to 390 kg/m²) results in a decrease in natural frequency by a factor of 1.45. Similar trends are observed for other values of self-weight. A key point is that when the frequency approaches 4 Hz, floor vibrations become significant and peak acceleration may exceed 0,5g so vibrations may become perceptible or uncomfortable, especially in offices or computer floors. At these frequencies resonance can occur with the human body's own natural frequency which amplifies the sensation of walking into an "unstable or bouncy floor". Moreover, when $f < 4$ Hz the structural capacity of the floor is nearly full used.

The optimal geometry for precast slabs and beams typically involves configuring the span of prestressed slabs to be between 1.0 and 1.5 times that of the supporting beams. This results effective because beams, which carry concentrated loads, face bending moments and deflections which are more sensitive to longer spans than slabs do.

In addition is advantageous for floor slabs to be recessed within the depth of the supporting beams, because this solution minimises the mass and contributes to a more efficient structural zone. So appears clear that finding the optimal orientation of the beams respect to the floor layout is a key part of the design.

The author Kim S. Eliott provides in his book [5] the orientation rule and some structural requirements: "the main beams should run parallel, or along the face of the longest building dimension, and the (mainly prestressed) floor slabs should span perpendicular to the beams and hence tie in to the building façade". Generally, this orientation pattern conflicts with the design approach used in cast in situ construction, where continuous beams typically span between internal columns perpendicular to the façade, or sometimes in both directions, supporting short, shallow slabs spanning either one-way or two-way between the beams.

When converting a beam-and-slab layout from cast in-situ to a precast solution, as illustrated in Figure 15, designers accustomed to in-situ methods may unintentionally adopt layouts like Plan A, compared to the more optimised Plan B. Plan A, for instance, consists of 33 beams, 28 columns, and 90 slabs (each 1.2 m wide), resulting in 151 components. Though Plan A's slabs are shorter, the same number of transportation and erection operations are required. By contrast, Plan B uses only 22 beams, 21 columns, and 60 slabs, just 103 components in total, resulting in 48 fewer pieces. Over a 10-storey building, this translates into 480 fewer erection operations. The structural requirements for Plan A and Plan B are given by the Author [5] in Table 2.

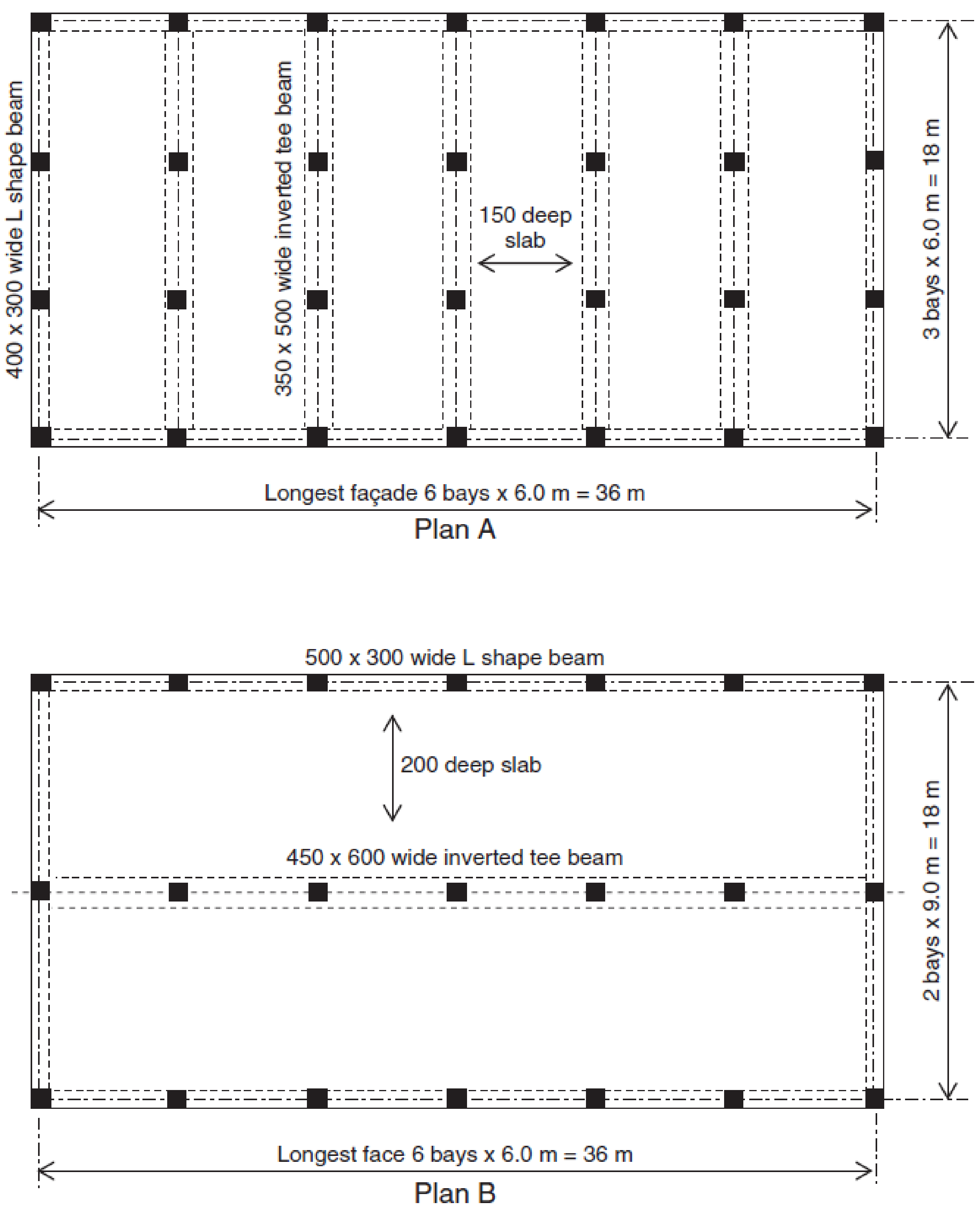

Figure 15 Alternative floor plans based on the orientation rule of floor and beam span directions.

	Plan A	Plan B	Differences
Clear span of floor slab (m)	5.50	8.55	
Depth of prestressed floor hcu (mm)	150	200	Plan B deeper floor but does not affect structural zone
Self weight of floor slab (kN/m^2)	2.7	3.3	Plan B 22% greater mass but 50% fewer units
Total floor service load (kN/m^2)	9.70	10.30	
Total floor ultimate load (kN/m^2) (γ_f = 1.25 (dead) and 1.5 (live))	13.38	14.12	Within 6% of each other
Clear span of beam (m)	5.70	5.70	Same, 6.000 – 0.300 m
Depth × breadth of inverted-tee beams (mm)	350 × 500	450 × 600	Plan B increase of 100 mm per storey
Ultimate load per internal column per storey (kN)	470	740	Plan B requires 400 × 300
Column size (assume five-storey) (mm)	300 × 300	400 × 300	Plan B 33% greater size but 33% fewer components
Floor bay area (m^2)	36.0	54.0	Plan B provides 50% greater area between column obstructions
Floor bay/structural depth ratio (m^2/m)	103	120	Plan B has 1.17 superior ratio in spite of deeper beams
Total weight of precast per storey	2425	2675	
Proportion of self weight due to floor slab	68%	77%	Plan B is greater due to deeper slab
Number of components per storey	151	103	Plan B 46% fewer

Table 2 Structural requirement for the alternative floor plans shown in Figure 15.

2.2.4 The Thermal Mass Period (2000 – present)

In the early 2000s precast concrete buildings gained attention thanks to their FES (fabric energy storage) properties which promoted them as energy efficient solutions. This era, often referred to as the "thermal mass phase," coincided with a growing architectural and engineering emphasis on sustainable design and energy efficiency. Concrete, especially in precast forms like hollow-core slabs and exposed wall panels, proved useful not just structurally, but as a passive tool for heating and cooling buildings.

The principle behind thermal mass is simple yet highly effective. During the day, exposed concrete surfaces absorb heat from sunlight and internal sources, helping to limit rapid increases in indoor temperature. As the air cools at night, this stored heat is slowly released, promoting a more stable indoor climate without heavy dependence on mechanical heating or cooling systems. This passive regulation is particularly advantageous in regions with significant temperature fluctuations between day and night. To enhance the effect, architects incorporated design strategies such as night time ventilation and exposed concrete soffits, enabling the building structure itself to manage thermal loads efficiently.

The response of the thermal mass of structural elements is quantified by “admittance”, the ability of an element to exchange heat with the environment when subjected to simple sinusoidal variation of temperature (example in Figure 16) over a given period (24 hours). Admittance is measured in $\frac{W}{m^2\,K}$, where K is the difference between mean daily temperature and the temperature at a point in time. The upper limit for naturally ventilated spaces is 8.3 $\frac{W}{m^2\,K}$.

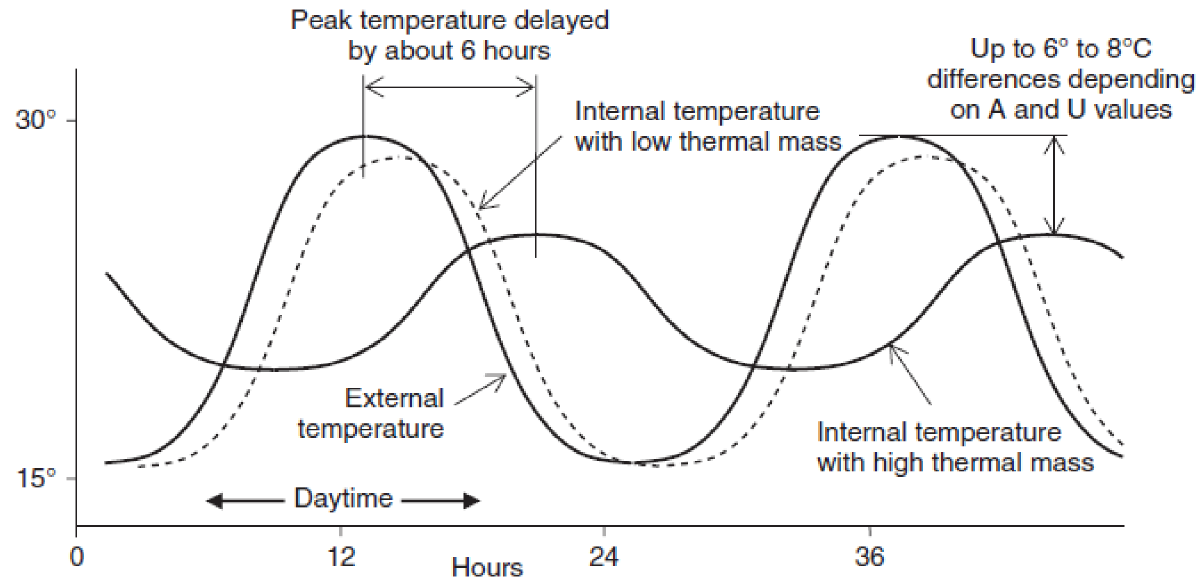

Figure 16 Stabilising effect of thermal mass on internal building temperature.

Table 3 lists values for common building fabrics, including precast concrete sandwich panels rated at 5.48 $\frac{W}{m^2\,K}$. This table combined with Table 4 shows the benefits of using precast concrete floors that have the higher cooling capacity without the need for water cooling according to [8].

Moreover, precast elements can be manufactured using White Portland cement, further reducing emissivity and lighting requirements.

One important aspect is that the described strategy is not universally applicable. The performance of thermal mass can vary significantly depending on the climate, building type, and usage. In regions with stable temperatures or in buildings that are occupied and conditioned continuously (without unoccupied night periods), the benefits are marginal. Another topic is as usual the correlation between structural-functional benefits with the architectural and aesthetic ones: the architectural requirements for exposing concrete surfaces sometimes argue with aesthetic goals, acoustic needs, or fire safety standards, forcing compromises in the design.

Wall construction	Admittance $W/m^2 °K$
Precast concrete sandwich-wall panel	5.48
Brick and dense concrete block cavity wall	5.75
Brick and block cavity wall	2.95
Internal block wall	2.09
Timber frame brick wall	0.86

Table 3 Admittance values for walls and facades.

Description	Application	Cooling capacity W/m^2	Benefits
Flat or profiled floor slabs with natural ventilation	Offices (with low internal gains), schools, universities	15–20 (flat) 20–25 (profiled)	No fan energy; minimal maintenance
Slab with raised floor with mechanical ventilation	Offices, public and commercial buildings	20–30 (flat) 25–35 (profiled)	Mixed mode ventilation system with windows; allows convective heat transfer with top of slab
Hollow conduit-core slab with mechanical ventilation*	Offices, schools, hotels, universities, theatres	40 (basic) 50 (+ supplementary cooling) 60 (+ switch flow)**	Large floor spans; air introduced at any level; well established technology
Water cooled slabs (plastic pipework)***	Offices, schools, hotels, universities	64 (flat) 80 (profiled)	High cooling capacity; water from boreholes, wells; good temperature control, precast option
Concrete slabs with chilled beams suspended below	Refurbished and new offices, as above	15 – 30 (FES†)	High cooling capacity; high chilled water tem-perature may allow free cooling; good temperature control

*Open-ended cores with controlled flaps may be used
** Flow may easily be reversed in these systems
*** *Pipe floor* or *Thermocast* are precast concrete options
† Chilled beams provide additional 100 – 160 W/m^2

Table 4 Cooling capacity for a range of FES and ventilated floor systems (based on [8]).

2.3 TYPOLOGIES AND APPLICATIONS IN THE CONSTRUCTION INDUSTRY

Precast concrete structures represent a significant innovation in the modern civil engineering, combing efficiency, quality control and enhanced structural performance.

This chapter will describe precast concrete structures across different hierarchical levels, in order to provide a comprehensive understanding of their behaviour and benefits following the imprint yet given by [9] (Figure 17).

The study will begin with an examination at the building scale to assess the different type of solutions present nowadays and to consider both the structural and the functional typologies. Structural typologies are based on technical and load bearing properties of precast concrete elements, while functional typologies are based on the spatial requirements of the intended use of the structure.

The focus then shifts to the component level, analysing the component subcategories, so the individual precast elements such as beams, columns, and slabs, followed by a brief exploration of the crucial role of connectors in ensuring stability, continuity, and the transfer of loads between components.

The chapter will finally conclude with a simplified structural frame analysis to demonstrate the key mechanical principles and validating the conceptual framework explained above.

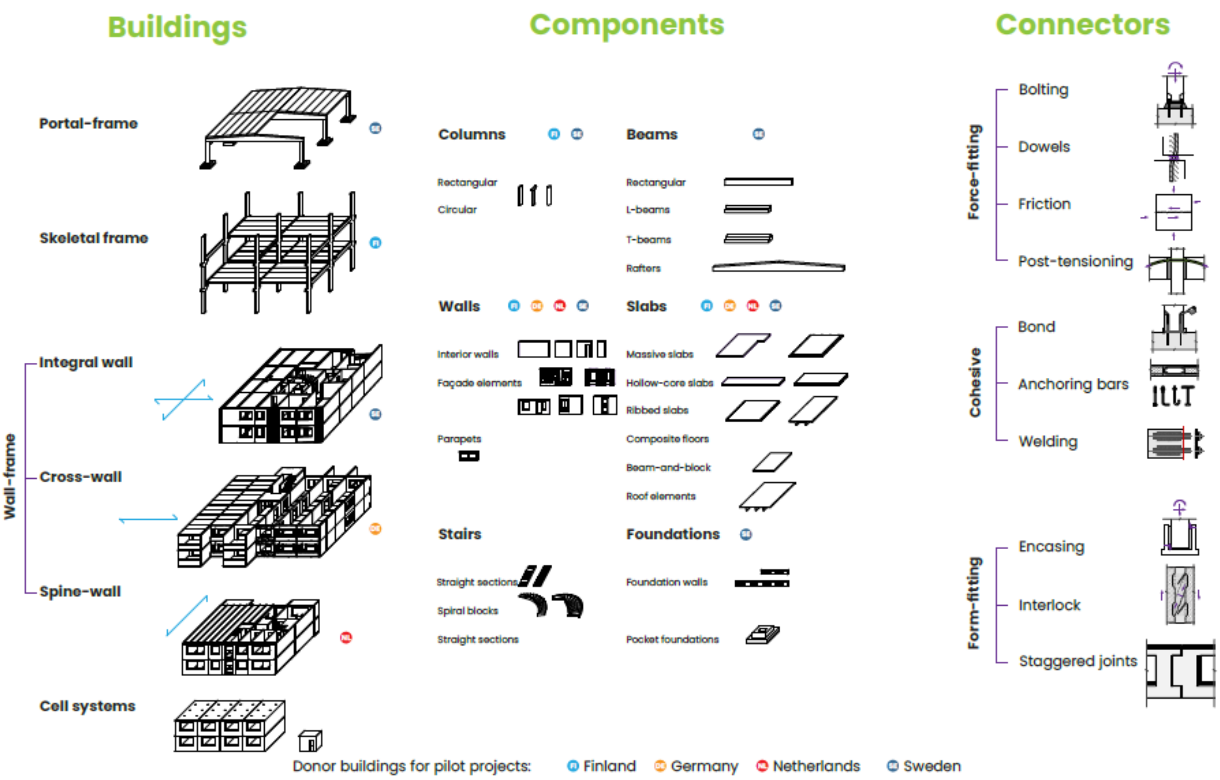

Figure 17 Main typologies at the building, component and connector scale.

2.3.1 Building scale

2.3.1.1 Structural typologies

Various structural systems utilizing prefabricated reinforced concrete elements have been developed over time. The prevalence of specific systems differs across countries, influenced by factors such as building purposes, the economic feasibility of various prefabricated components, and the design philosophy and creativity of individual designers.

Prefabricated structural systems generally fall into the following categories (Figure 18):

- Portal frame structures;
- Skeletal frame structures;
- Wall frame structures;
- Cell structures.

The structural system of a specific building may rely on a single basic system or on the combination of these. Additionally, the system can be composed entirely of prefabricated components or a combination of prefabricated elements and cast-in-place concrete, these are known as mixed solutions [10].

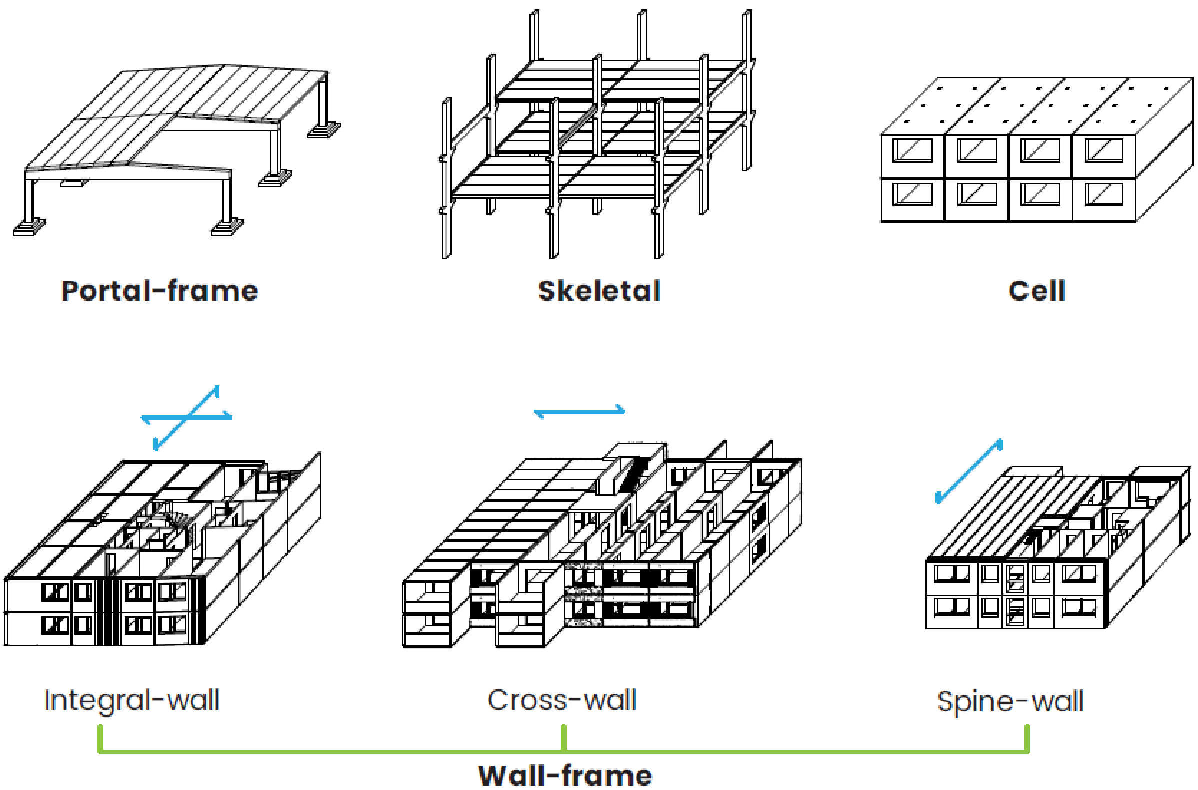

Figure 18 Main structural typologies for precast concrete systems.

Portal frame structures

Portal frame precast structures are commonly used in industrial and commercial construction due to their simplicity, rapid assembly process and high structural efficiency. These systems are peculiar for warehouses where spans of 25 - 40 m I sections or T sections rafters are necessary as in Figure 19 and Figure 21. Portal frames are rarely used for residential buildings but may actually be used to form the roof of a skeletal frame.

These portals are generally made of beams and columns arranged to form rigid or semi rigid frames capable of spanning significant distances without the need for intermediate supports. The columns are generally designed as moment resisting cantilevers, this means that are rigidly anchored at the base, while the beams are jointed at the top using moment resisting or pinned connections. Generally, moment resisting connections are really demanding to be created on site and so the latter ones are preferred due to ease of design and construction. A key point for these hinged connections is to take in consideration that the flexural rotation at the ends of the main rafters can lead to cracking damage at the bearing ledge, this is commonly mitigated through the use of a flexible pad, such as neoprene, at the bearing.

In some situations, a moment resisting connection is employed allowing for some degree of continuity for the moment at the eaves where column and beam meet, but, unless the columns exceed 8 m height the additional complexity of the connection may not justify the effort.

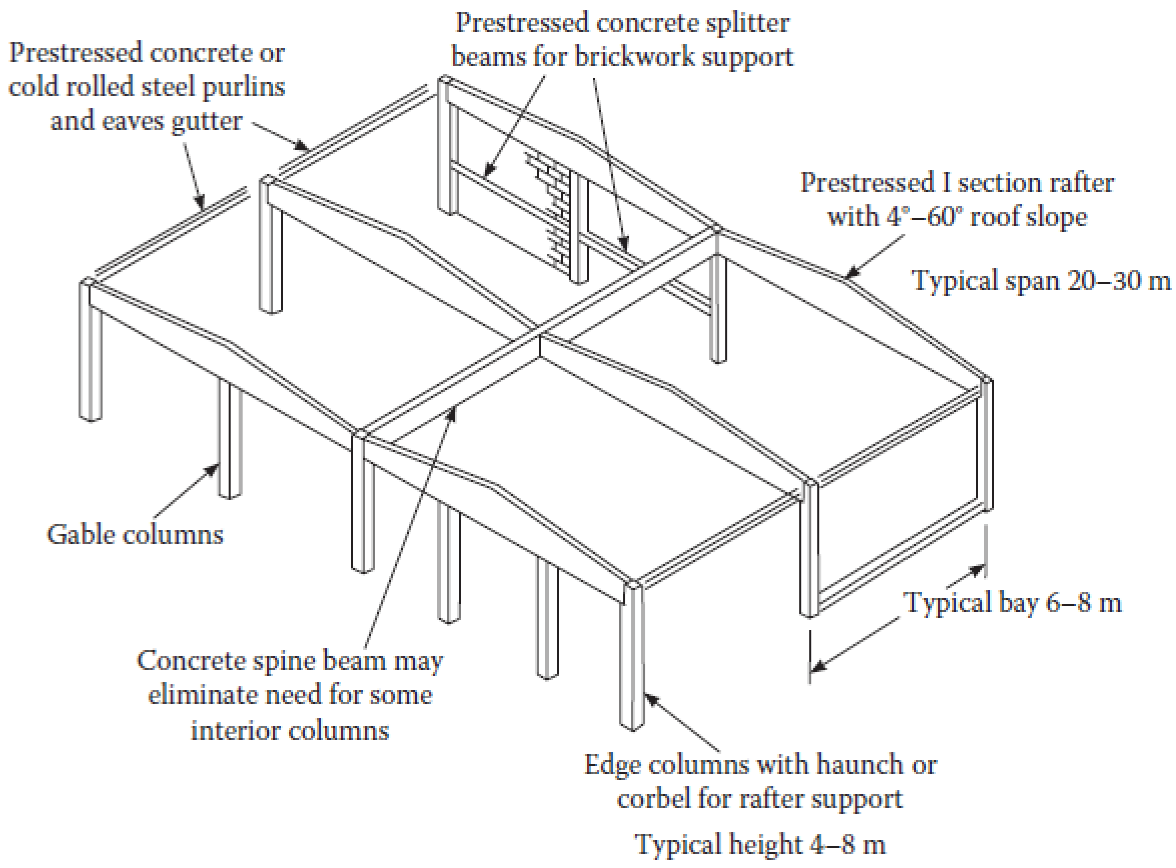

Figure 19 Definition of a precast portal frame.

Figure 21 Precast portal frame.

Figure 20 Portal frame with folded plate roof units, the University of Sao Carlos, Brazil.

One of the most used options for industrial buildings is the precast portal frame with flat or shallowly inclined roof structures composed by beams spanning 6 - 8 m which support long span folded plate roof elements of around 20 m length. As illustrated in Figure 20 can be additionally inserted in the design overhang beams for shading against sun or rain.

Skeletal frame structures

Skeletal structures are composed by a network of columns and beams to constitute a three-dimensional frame that supports the building. The main aim of those is to create large open plan accommodations and so give a great variety of possibilities for space utilization. (Figure 22)

They are particularly suitable for office and retail development, where distances between columns and beams are usually in the range 6 - 12 m depending on the loading conditions and on the intended use; and for multi storey car parks where floor spans can reach values up to 16 m.

These structures consist into multiple elements that interact to form a stable framework:

- Precast columns transfer the loads from the superstructure to the substructure, resist to compression, bending moment and shear forces due to lateral actions. They can have different type of cross sections depending on the intended use and required performances and can be manufactured with corbels or inserts for the connection with beams and slabs.
- Precast beams are horizontal elements that connects the vertical ones to form a rigid frame. They transfer the load from floor slabs to columns and participate in lateral load resistance. The cross section and the depth are governed by the span length and loading requirements.
- Floor systems can be either precast slabs or cast in situ slabs. In any case they transfer dead-loads and variable loads to the beams and provide additional stiffness for the lateral load resistance of the frame.

Figure 22 Precast skeletal structure highlighting large open plan accommodation.

The connections between the different elements are a crucial aspect for load transfer capacity. From a structural point of view the connections can be rigid or pinned [11].

Rigid connections are really demanding and are used to transfer bending moment and so create rigid frames, they can be achieved with the continuity of reinforcement through the beam column connection by welding the bars together or using mechanical couplers (Figure 23).

Pinned connection are in generally preferred for simplicity of construction and design and are achieved by seating the beams on column corbels and by holding the beam ends in place by welded steel shoes or using vertical dowels, so that the shear can be transferred but not the bending moment. (Figure 24)

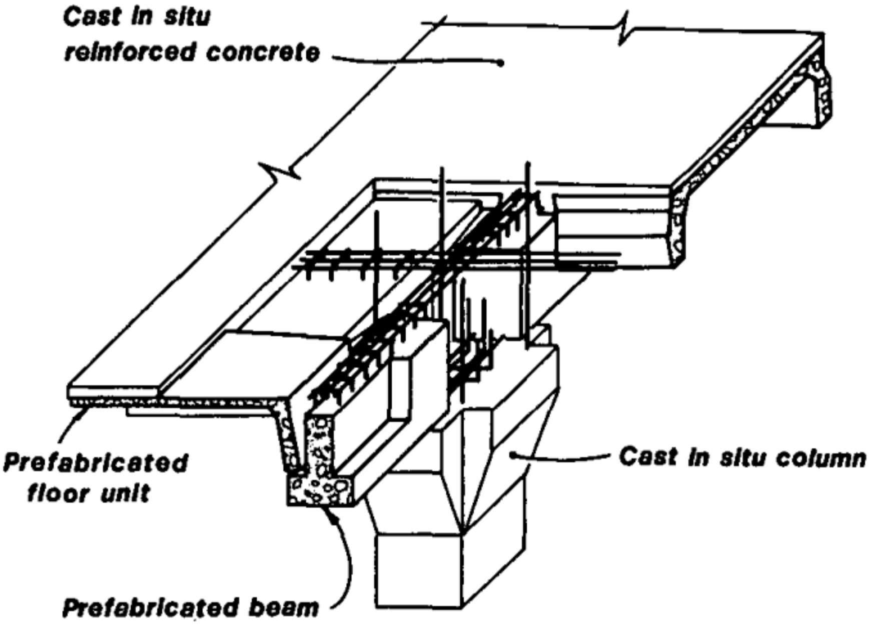

Figure 23 Example of a rigid connection in a frame system.

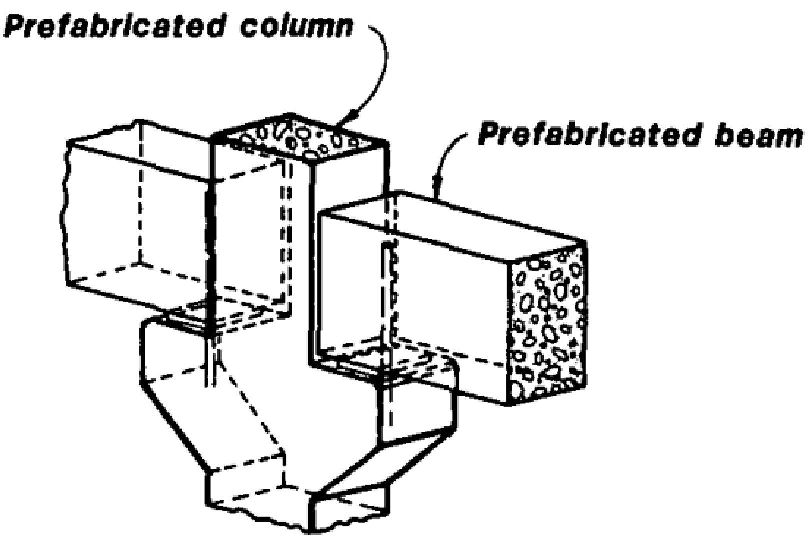

Figure 24 Example of a hinged connection in a frame system.

Wall frame structures

Wall frame precast concrete structures consist on vertical wall panels that act as main load-bearing components, fulfilling both structural and architectural roles. These large panels support vertical loads and, depending on the design, also horizontal loads from wind or earthquake actions.

Wall frame systems or large panel systems are used especially in residential buildings. This type of construction may be faster to be erected but leaves less architectural freedom respect to the skeletal frames because the panels should be arranged following a fixed grid or a fixed modular distance (usually 3 m) which is at the base of the economy of the process. The distance between walls strictly depends on the building purpose varying from 6m for hotels, schools, offices to 10 - 15 m for commercial constructions.

The structural analysis of these panels is not as simple as seems because they have a very large in plane rigidity while the connections between walls and floors are very rigid; moreover may arise differential settlements between wall panels and between walls and floor slabs that can result in serviceability problems over the life of the structure.

The walls are typically connected both in the horizontal and vertical direction with precast elements like slabs and beams to create a rigid box-shaped framework (Figure 25). Depending on the direction of the joint it’s possible to have:

- Vertical joints, which connect the vertical edges of two adjacent joints and primarily resist to vertical shear forces.
- Horizontal joints, which connect the horizontal edges of adjacent wall and floor panels and primarily resist vertical normal forces due to gravity loads from the upper panels and floors, horizontal shear force due to seismic loads, and bending moments in two directions due to seismic loading acting on the upper panels and gravity loading acting on the adjoining floor panels.

In general the joints may be dry or wet. Dry joints are constructed by welding or bolting together steel plates which have been cast into the ends of the prefabricated panels for this purpose and make a quick erection possible. Wet joints are done with cast in situ concrete in the edge regions between panels and result in a structure more closely approaching monolithic construction.

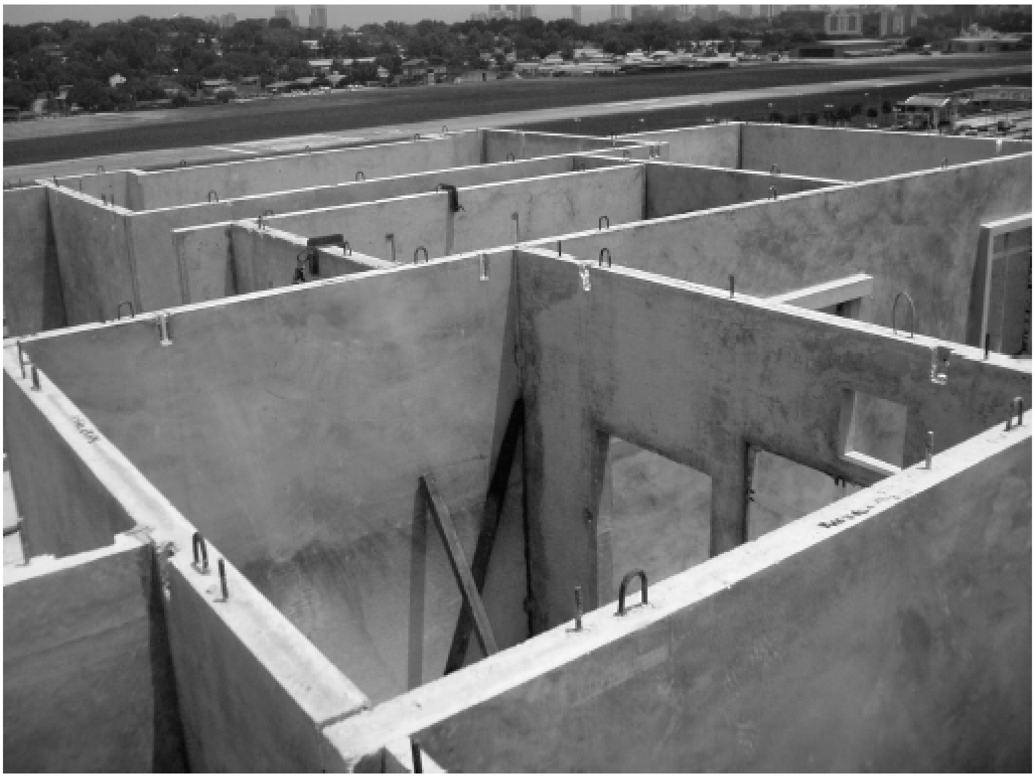

Figure 25 Cross-wall system in precast wall frames.

For wall frame structures can be distinguished three basic configurations [12]:

- Cross-wall system. In this arrangement, the walls that carry gravity loads are oriented perpendicular to the building's longitudinal axis, as depicted in Figure 26a. These cross-walls not only support one-way spanning floor or roof components but also resist horizontal seismic forces along their direction. Walls that do not bear gravity loads are aligned parallel to the longitudinal axis, serving to resist horizontal seismic forces in the corresponding direction.
- Long-wall system. Here, walls bearing gravity loads run parallel to the longitudinal axis of the building, as shown in Figure 26b. These long-walls resist horizontal seismic loads along their line and also sustain gravity loads from one-way spanning floor or roof components. Non-load-bearing walls, placed perpendicular to the longitudinal axis, provide resistance against horizontal seismic forces in the transverse direction.
- Two-way system. In this configuration, walls supporting gravity loads are arranged both perpendicular and parallel to the longitudinal axis, as illustrated in Figure 26c. These walls effectively resist horizontal seismic loads in both directions and accommodate gravity loads from two-way spanning floor or roof elements.

In addition, we can have also the spine wall systems. In these configurations the slabs are oriented in transverse direction of the building and the spine walls are needed along the longitudinal axis of the structure because generally the width of the building exceeds the span capacity of the floor elements.

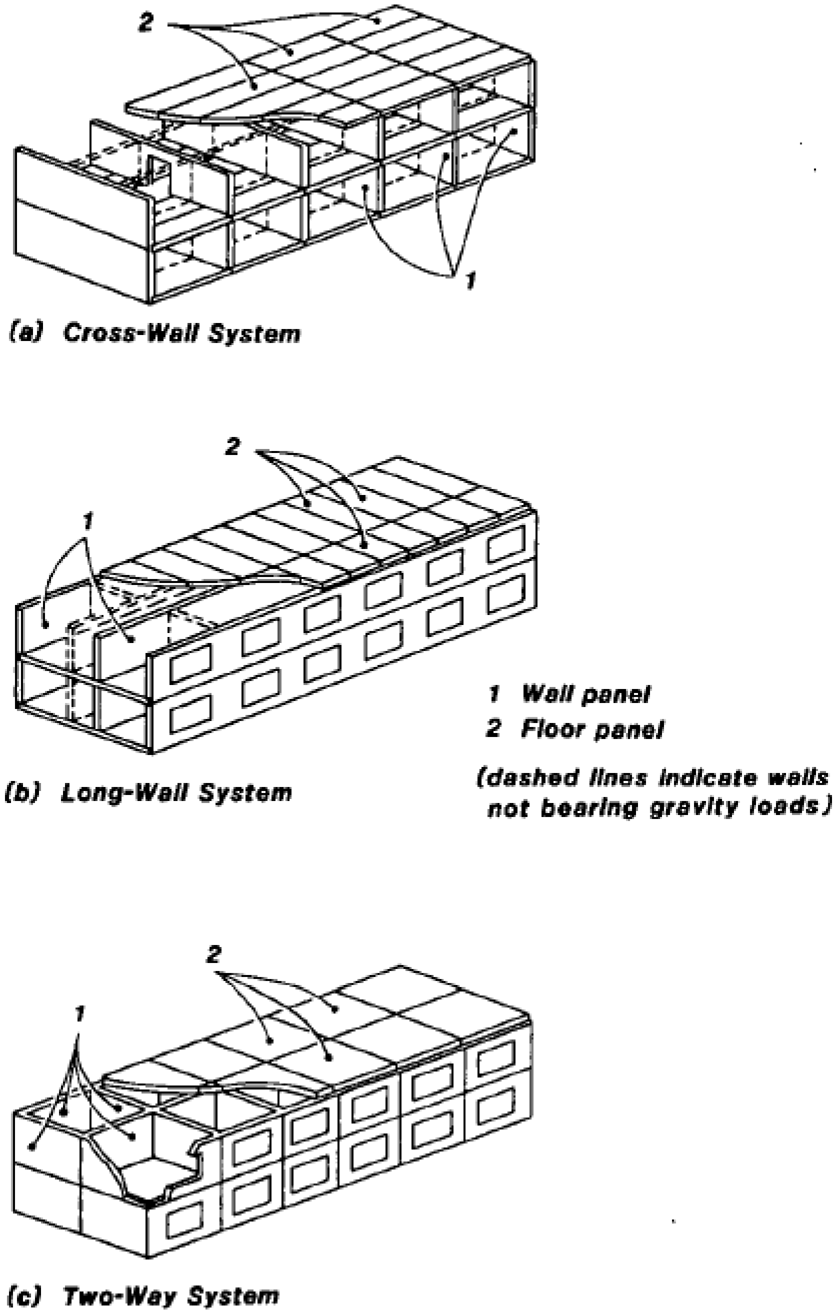

Figure 26 Basic Structural Configurations of wall systems.

The following table (Table 5) is useful for a summary of these types of precast structures considering their possible application.

Use of building	*Number of storeys*[a]	*Interior spans (m)*	*Skeletal frame*	*Wall frame*	*Portal frame*
Office	2–0	6–15	✓		
	2–50	6–15		✓	
Retail, shopping complex	2–10	6–10	✓	✓	
Cultural	2–10	6–10	✓		
Education	2–5	6–10	✓	✓	
Car parking	2–10	15–20	✓		
Stadia	2–4	6–8	✓		
Hotel	2–30	6–8		✓	
Hospital	2–10	6–10		✓	
Residential	1–40	4–6		✓	
Industrial	1	25–40			✓
Warehouse with office	2–3	6–8	✓		
		25–40			✓

[a] Typical values, depending on the location, terrain, requirements, etc.

Table 5 Application and types of precast concrete frames.

Cell systems

The box or cellular system, closely associated with the wall system in its final erected form, offers a distinctive construction approach (Figure 27). Concrete boxes can either be cast as single integral units or assembled from individual components connected to function as a cohesive structure. This system generally involves producing off site room-sized prefabricated box units and assembling them vertically on site. These walls resist lateral seismic forces in both directions and support vertical gravity loads.

Cell units can be used either to build complete structures and for specific sections of a building, such as bathrooms or technical units.

Figure 27 Construction with cell systems moved with a crane.

2.3.1.2 Functional typologies

The distinct technical characteristics of various structural systems discussed in the previous section are closely tied to the specific functional needs of the building type, such as residential, commercial, industrial, or office structures. These building typologies can be categorized using different criteria. In this study is proposed the classification made in [9]: a hierarchical classification system with two primary branches which are buildings and infrastructure (Figure 28). On the other hand, is also possible to associate the functional typology to the structural system in order to get a complete view of the actual development of precast structures (Figure 29).

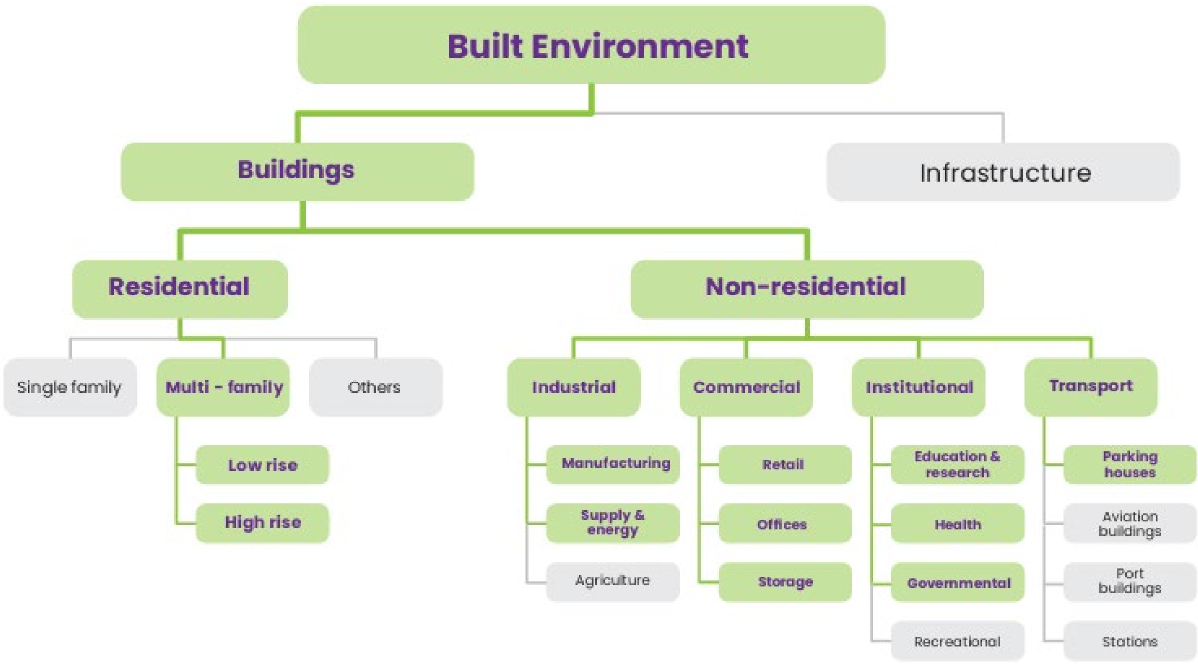

Figure 28 A hierarchical representation of the built environment highlighting the typologies most relevant for ReCreate.

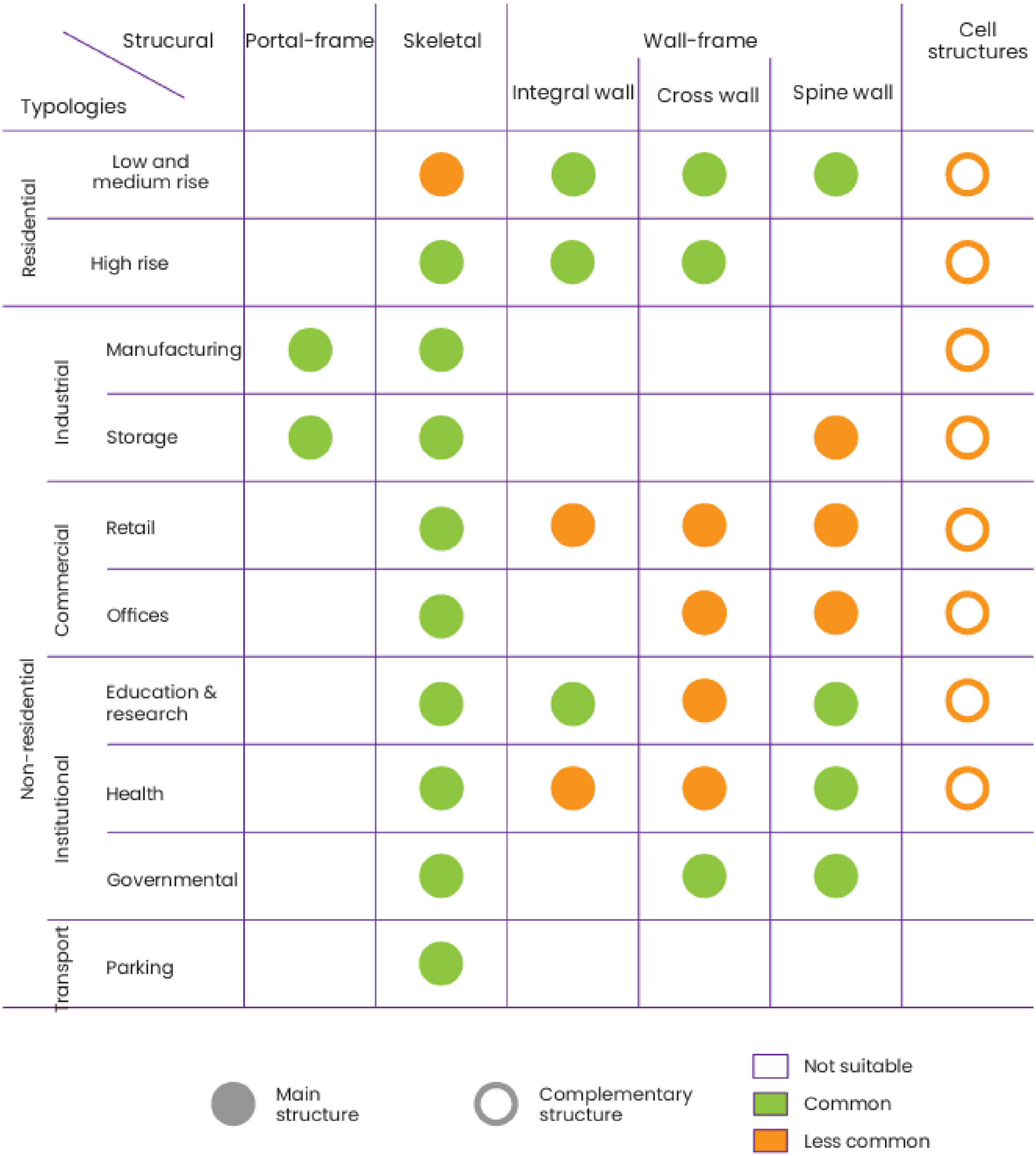

Figure 29 Correspondence between building typologies and structural systems.

2.3.2 Component scale

A further classification is at the level of the component scale, precast concrete elements (PCE) are initially ordered based on their geometry and proportions, separating them into block, panel and linear elements.

Huge components as slabs and walls with two long dimensions are commonly referred as panels, components with one predominant dimension such as beams are called linear elements and thicker and shorter components with two comparable dimensions are defined as blocks. (Figure 30)

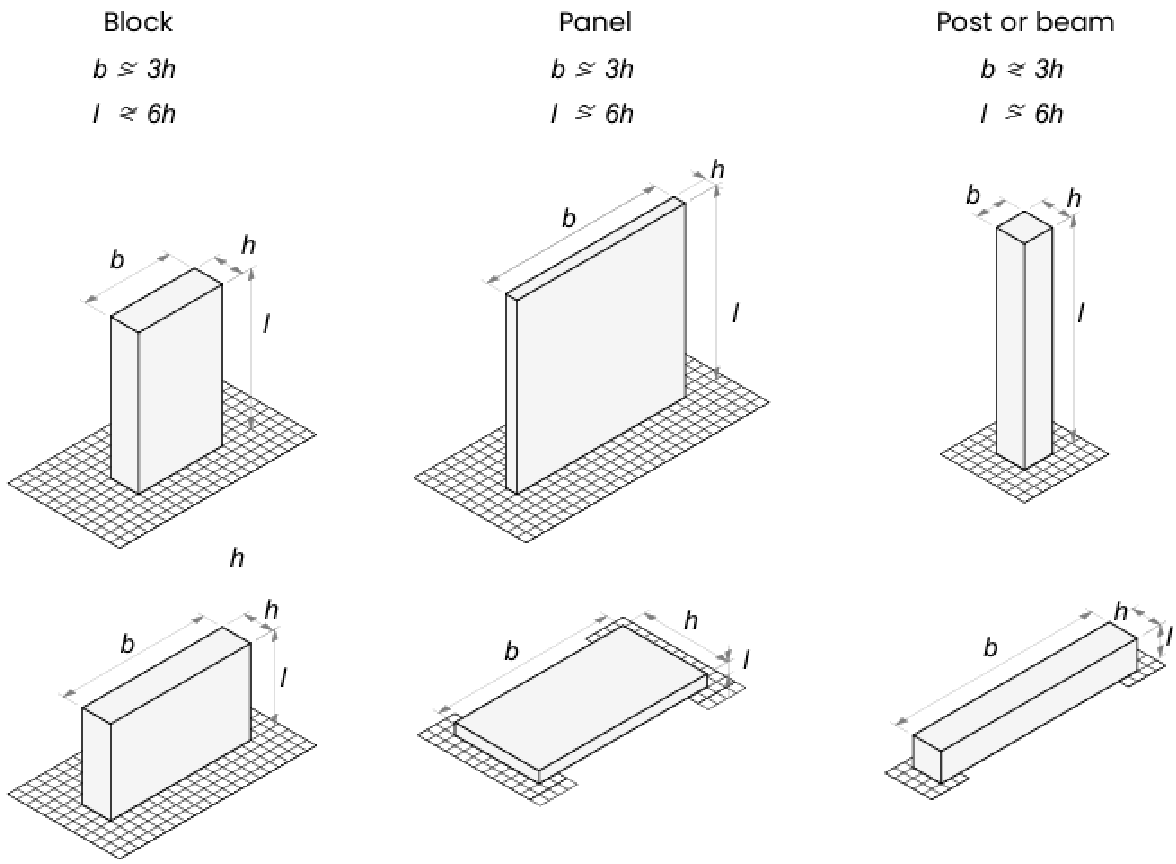

Figure 30 Classification of precast concrete elements according to their proportions.

A more general classification can be obtained by considering the subcategories for each PCE described above, in order to meet structural roles and architectural requirements.

Columns

Precast concrete columns are fundamental structural components that play a crucial role in supporting and stabilizing buildings by efficiently channelling vertical loads from upper structural elements, such as beams and slabs, down to the foundation. Engineered to fulfil both strength and serviceability requirements, these columns ensure the long-term reliability and durability of a structure throughout its lifecycle. Produced off-site under controlled conditions, precast columns deliver superior quality control, faster construction timelines, and reduced on-site labour demands compared to cast-in-place alternatives. Available in a range of shapes and sizes, precast concrete columns include standard rectangular and circular cross-sections as well as architecturally more complex designs to address aesthetic or spatial needs. The selection of column dimensions and profiles depends on factors such as structural load capacity, fire resistance, connection detail demands, and architectural integration. Typically, a minimum cross-sectional dimension of 300 mm is employed to achieve structural adequacy while accommodating reinforcement detailing, beam-column connections, and construction tolerances. For enhanced fire resistance, increasing the cross-sectional dimension to 400 mm can provide up to two hours of fire protection. The height of precast columns can vary considerably depending on the structural system and project type. In large-scale structures such as industrial facilities and high-rise buildings, precast columns can reach heights of 24 meters or more. Such tall columns often incorporate advanced reinforcement techniques like post-tensioning or high-strength concrete to enhance load capacity and control cracking. On the other hand, single-storey precast columns are more common in commercial and residential constructions due to their ease of transport, installation, and compatibility with modular building systems. Equally important is the integration of connection systems, such as grouted sleeves, mechanical couplers, or embedded plates, which are essential for ensuring load transfer and continuity between different structural components.

Beams

Beams, as depicted in Figure 31, serve as essential horizontal structural components, specifically designed to support and distribute loads from slabs or roof elements and transfer them to vertical structural elements like columns or walls. In contemporary construction, precast concrete beams are widely used due to their enhanced quality control, quicker installation process, and superior ability to span greater distances compared to cast-in-place variants. These attributes make them especially advantageous for projects prioritizing rapid construction, consistent replication, and long, uninterrupted spans, making them ideal for commercial, industrial, and institutional buildings. Precast beams are versatile in shape and are manufactured in a variety of cross-sectional profiles tailored to particular structural and architectural needs. The rectangular beam (Figure 31a) remains the most commonly used type due to its easy design and compatibility with standard framing systems. For applications that demand improved space utilization and performance, shapes such as L-shaped beams (Figure 31b) and inverted T-beams (Figure 31c) are commonly used. These configurations efficiently support slabs while minimizing beam depth, thereby reducing the overall floor-to-floor height. For scenarios requiring extended spans or higher load capacities, common in portal frames, skeletal structures, and industrial facilities, more complex profiles like I-beams (Figure 31d) are typically chosen. These beams offer an excellent strength-to-weight ratio, optimizing material usage while meeting demanding structural requirements. Another specialized type is the rafter beam (Figure 31e), which features a variable depth along its length to match the slope of pitched roofs or focus material thickness where structurally needed. To achieve optimal performance, precast beams are often reinforced with prestressing tendons or high-strength reinforcement to boost their bending capacity while controlling deflections and cracking under load conditions. Connection options include corbel supports, pocket foundations, and grouted sleeve systems, each engineered to facilitate effective force transfer, ensure stability against seismic or wind loads, and simplify on-site installation.

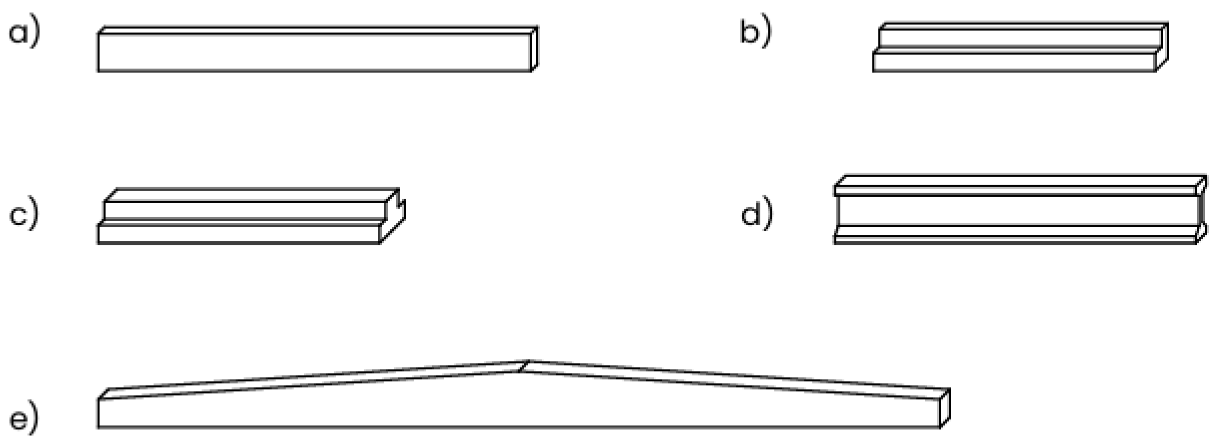

Figure 31 Subcategories for precast beams.

Walls

Precast concrete walls, as illustrated in Figure 32, are versatile building components capable of serving both structural and non-structural roles within a construction system. Structurally, these walls can work as load-bearing elements, effectively transferring vertical and lateral loads down to the foundation, thus playing a crucial role in ensuring the stability and seismic resilience of the structure. On the non-structural side, they are often used as cladding panels or curtain walls designed to enhance aesthetic appeal and provide environmental benefits such as thermal insulation, weather resistance, and soundproofing. These wall elements generally fall into two primary categories: interior walls, which can act as partitions or load-bearing divisions, and exterior walls, which form part of the building's envelope. Precast walls may come as single-layer units, sandwich panels with built-in insulation, or more complex multi-layered assemblies that integrate mechanical and electrical systems. The high level of adaptability in precast wall production allows for the inclusion of features like openings for windows and doors, pre-installed conduits and utility boxes, thermal insulation layers, and a variety of surface finishes. While window and door opening dimensions can be customized to meet specific design requirements, many manufacturers prefer standardized dimensions to simplify the production process, lower costs, and reduce material waste. Regardless of the dimensions chosen, it is critical to ensure minimum thicknesses and widths for structural elements like lintels especially for those positioned between adjacent openings or near panel edges. Adhering to these minimum specifications is vital to maintain the structural integrity of the panels throughout key phases such as demoulding, transportation, and on-site installation. Insufficient dimensions can lead to cracking or failure due to the stresses encountered during handling and placement.

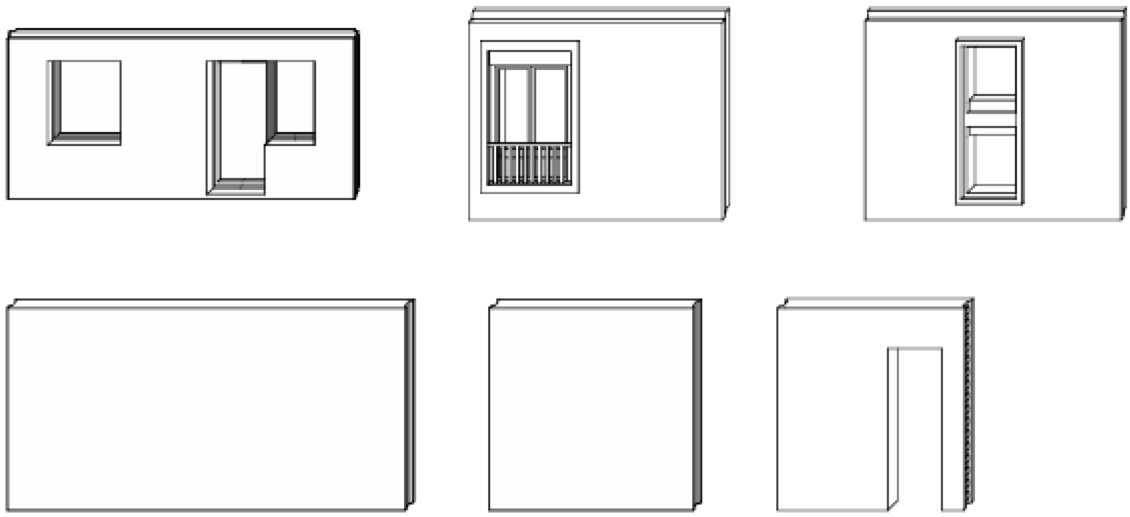

Figure 32 Examples of precast wall elements.

Slab and roof elements

Precast floor and roof systems (Figure 33) play a vital role in contemporary construction by delivering both structural efficiency and faster installation timelines. These systems are often defined as sub-systems within the larger structural framework and are divided into two categories: fully precast systems and semi-precast systems. Fully precast systems are completely produced in the factory and then are transported to the site ready for installation. This process minimizes the need for on-site concrete work, improves quality control, reduces labour requirements, and significantly accelerates construction. On the other hand, semi-precast systems use a combination of precast elements that are complemented with in site cast concrete. This hybrid technique offers the benefits of both systems: precision and speed of precast methods with the continuity, strength, and monolithic behaviour of cast-in-place concrete. Precast concrete flooring options come in various types, each tailored to meet distinct structural, architectural, and functional requirements:

- Massive slabs (Figure 33a): Solid precast panels designed for applications demanding high load-bearing capacity or superior acoustic insulation.
- Hollow-core slabs (Figure 33b): Lightweight, prestressed units with longitudinal voids that reduce self-weight while maintaining strength, ideal for long-span floors in residential and commercial properties.
- Ribbed floors (Figure 33c): Systems featuring closely spaced ribs to enhance load distribution and lower dead weight. These are particularly suitable for industrial facilities and parking structures.
- Composite floors (Figure 33d): Systems combining precast elements with a structural topping slab that binds with the precast unit to form a unique section. These are commonly used in offices or educational buildings, especially where integrated mechanical, electrical, and plumbing services are required.

- Beam-and-block systems (Figure 33e): Modular flooring solutions utilizing precast beams and infill blocks, often applied in low-rise residential projects due to their cost efficiency and ease of assembly.

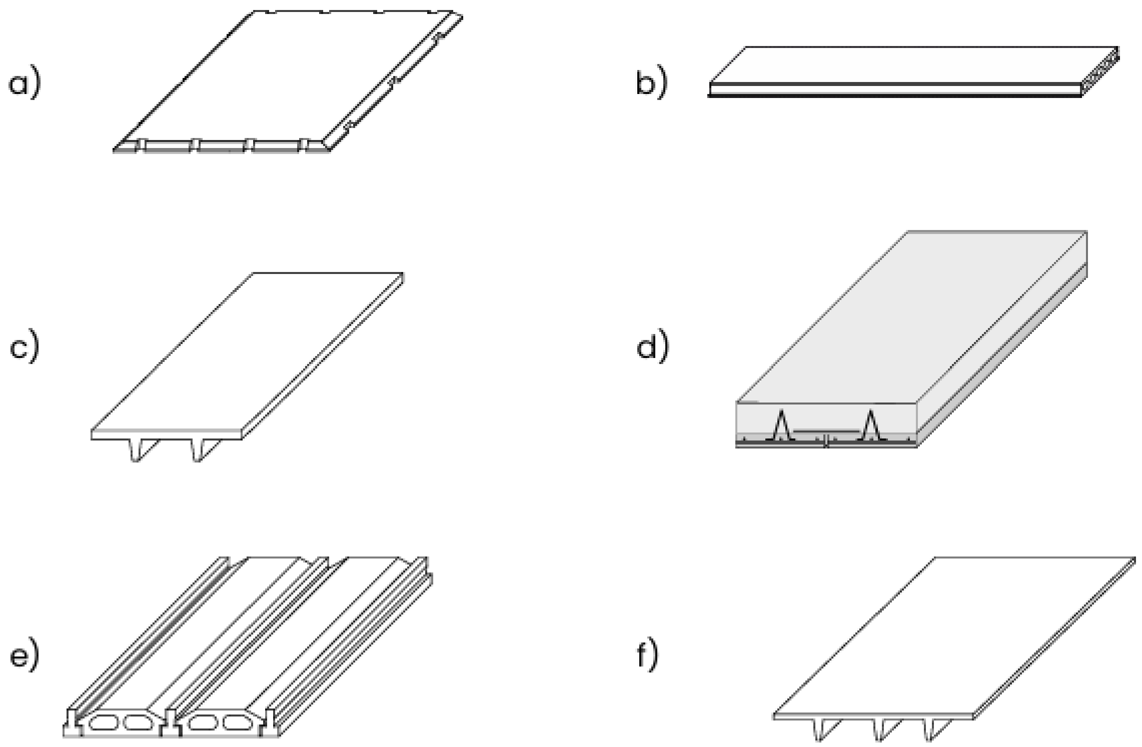

Figure 33 Subcategories for precast slab and roof elements.

To optimize manufacturing and installation processes, these systems are typically designed using standardized modular widths, often set in 300 mm increments. Common panel widths include 600 mm, 1200 mm, and 2400 mm, ensuring efficient transport, handling, and site placement. These dimensions also support modular coordination with other structural elements such as beams, columns, and walls. Beyond their structural advantages, precast floor and roof systems provide added benefits such as fire resistance, acoustic insulation, and thermal mass. These features can be further enhanced through specific choices in concrete mix design, panel thicknesses, or integrated insulation materials.

Stairs

Precast stairs are widely used in residential, commercial, and public buildings due to their durability, rapid installation process, and smooth finish. Stair components are available in various configurations tailored to specific project needs. The most prevalent type is the straight stair element (Figure 34a), which can be produced with integrated landings as a single continuous unit or with detached landings that are assembled on-site. Detached systems facilitate easier transportation and allow for greater flexibility during installation. These straight elements are ideal for standard floor-to-floor transitions and are commonly designed to align precisely with precast floor slabs or wall panels. More intricate precast stair designs include monolithic stair modules that consolidate multiple elements into a single casting. Examples include spiral or helical stairs (Figure 34b), and monoblock stair units (Figure 34c). For safe handling and effective installation, precast stair units are engineered with integrated lifting anchors and detailed connection points, enabling secure integration with structural elements like precast walls, cores, or steel frameworks. Additionally, the undersides of the stair units can be customized to accommodate mechanical or electrical systems.

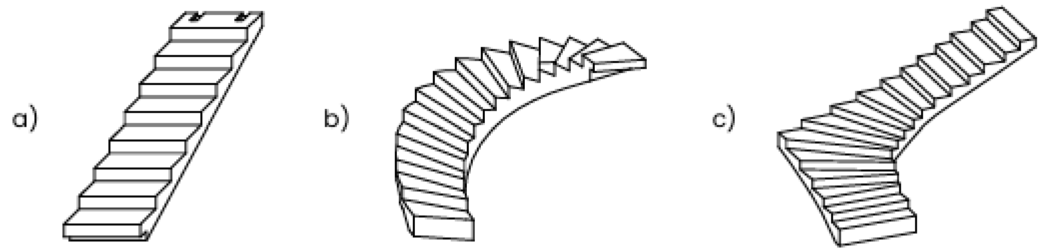

Figure 34 Subcategories for precast stairs.

Foundation elements

Foundation components, including piles, footings, and grade beams, play an important role as the link between a building and the ground. They ensure that structural loads are effectively and safely transferred to the underlying soil or rock layers. Precast concrete foundation systems have gained significant importance as an efficient alternative to traditional cast-in-place methods. A notable example is pocket foundation elements, designed with recessed spaces or pockets to accommodate precast columns or walls. These systems are particularly suitable for regions with stable soil conditions, facilitating rapid on-site assembly, enhanced alignment accuracy, and immediate load transfer, eliminating delays caused by curing or formwork removal. Beyond pocket foundations, precast footings and grade beams are also produced off-site for quick installation. This minimizes on-site disruption while accelerating construction timelines. This method proves especially beneficial for projects with repetitive layouts, such as warehouses, modular housing developments, or agricultural facilities. Precast foundation walls offer additional advantages, especially in cold climates where extended curing periods and weather constraints often make more complex cast-in-place construction. Prefabricated foundation walls can be installed regardless of temperature, significantly shortening construction schedules. Furthermore, these walls can incorporate features such as insulation layers, built-in service conduits and drainage systems, improving both their thermal performance and overall ease of construction.

2.3.3 Connector scale

Connectors are integral to the functionality and stability of prefabricated structures, serving as key components that ensure the consistent and reliable transfer of forces among individual elements while integrating them into a cohesive structural system. Moreover, they must also accommodate additional stresses arising from thermal displacements, shrinkage, creep and fire. An important point for the reuse of precast structures, is that the design of connectors significantly influences how easily the elements can be separated and removed without causing damage.

Additionally, connectors contribute to fulfilling both functional and aesthetic requirements, such as ensuring durability and visual appeal. The detailing of these connectors must also consider practical factors like the manufacturing, transportation and assembly processes associated with precast elements.

In the context of precast structures, connections are primarily divided into two types: permanent connections that remain throughout the structure's service life and construction connectors used only during the assembly phase to lift or temporary secure the elements in the construction stages. Permanent connectors are then further classified in two subtypes: wet and dry connections. Wet connections rely on materials such as concrete or grout to bond elements together, whereas dry connections use mechanical methods for joining.

Figure 35 is taken from [9] and highlights the various possibilities and combinations of connections between different structural elements: the most common connections are represented in green while the less frequent ones in yellow. Some spaces appear empty because are cases that typically correspond to elements that are not structurally linked, such as columns and walls belonging to separate structural systems, like skeletal and wall-frame systems.

A further step is made in Figure 36 where are presented the different types of components and the connections commonly employed to join them, which will be further analysed in the following sections.

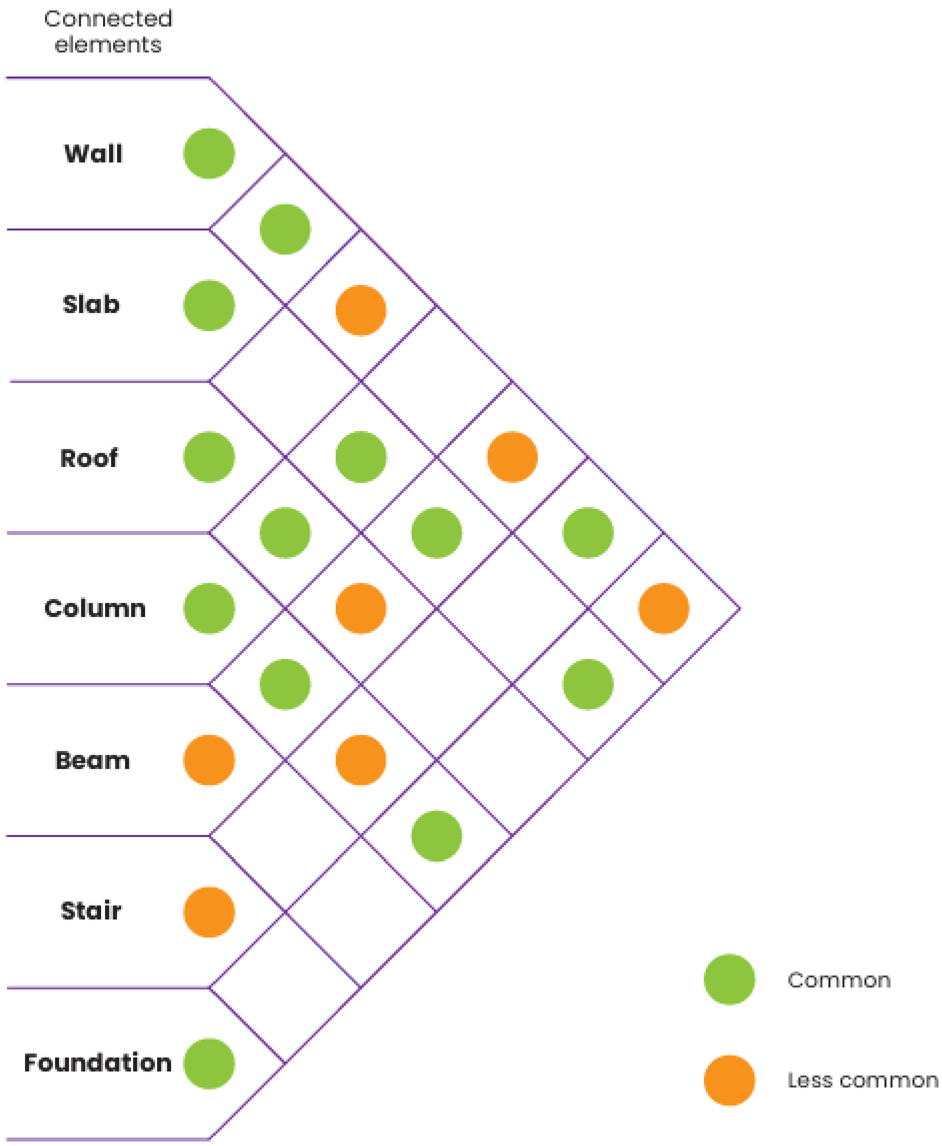

Figure 35 Diagram showing the most common component-to-component connections.

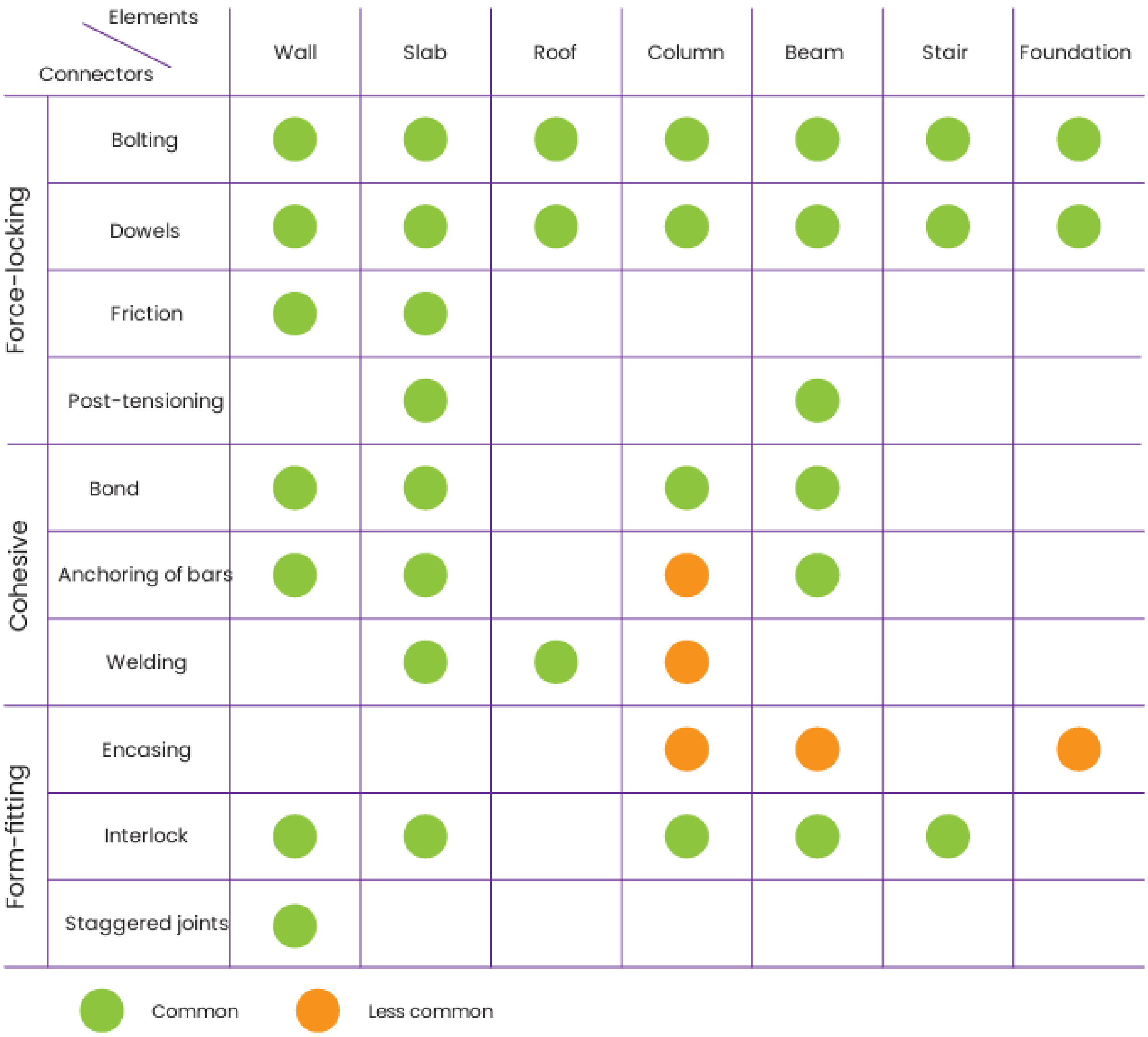

Figure 36 Classification of connector types and elements involved.

The force transfer mechanism is used to classify the connections into three broad categories: force-locking, cohesive and form-fitting. Force-locking connections rely on the application of forces, such as friction, tension, or compression, to secure the connection between components. Cohesive connections, on the other hand, are achieved through material bonding, creating continuity using methods like adhesives, welding, grouting, or similar techniques that merge the elements. Form-fitting connections depend on interlocking geometries to restrict movement between parts, eliminating the need for friction or adhesive forces. It is worth to notice that these mechanisms for force transfer are often combined in various ways to meet specific connection requirements. For instance, connections between wall and floor elements typically integrate both friction and dowel action.

Force-locking connections

In force-locking connections, as illustrated in Figure 37, the components are secured primarily through the frictional forces generated between their contacting surfaces. This effect is typically achieved by applying an external force, such as in bolted or clamped assemblies, where the tight fit produces the necessary friction to maintain the connection. Such connections are capable of withstanding forces directed in specific ways, depending on the nature and intensity of the applied force.

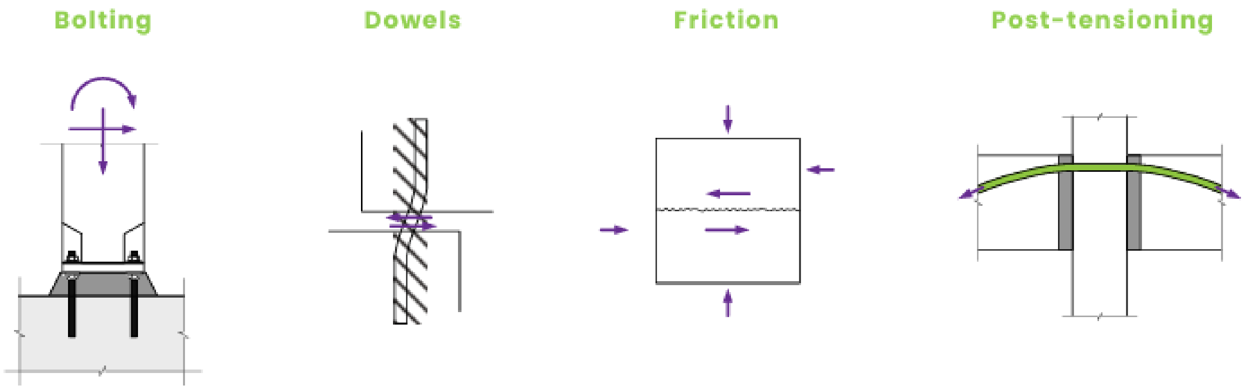

Figure 37 Transfer mechanism based on force locking.

Different types of force-locking connections are present nowadays:

1. Bolting connection is a widely employed technique for connecting precast elements, designed to efficiently handle tensile and shear forces. This method relies on various anchoring systems, including bolts, threaded sockets, rails which are integrated into the precast components. As illustrated in Figure 37, a typical application is the bolted connection between a column and its foundation. These anchoring systems create a mechanical interlock that effectively withstands applied loads. To account for construction tolerances, oversized holes are often incorporated into the connecting members, enabling adjustments during assembly. A significant advantage of bolting lies in its simplicity and the speed of installation and removal, making it particularly well-suited for temporary structures, projects requiring future modifications, or situations where reduced acoustic and fire-resistance criteria are acceptable. The strength and performance of bolted connections are largely influenced by the quality of the anchoring components, the bolt material and the precision during installation.
2. Dowels are steel bars partially embedded in concrete and able to transfer shear forces across joints in precast structures. This behaviour resembles that of a beam supported by an elastic foundation, with the steel bar acting as a dowel pin. It experiences variable concrete stresses and localized bearing forces, particularly close to the joint interface, as shown in Figure 37. The dowel undergoes both shear forces and bending moments, potentially leading to different failure modes based on the relative strength of the bar and its placement within the concrete element. For instance, a weaker bar may fail due to shear, while a stronger bar embedded in less robust concrete or positioned with insufficient cover could result in splitting of the surrounding concrete. An efficient distribution of loads and a correct amount

and position of reinforcement are crucial to ensure the effective transfer of shear forces and the integrity of the connection.

3. Friction-based connections exploit the roughness at joint interfaces to transfer shear forces, relying on compressive stresses at the interface to produce adequate friction resistance, as illustrated in Figure 37. In cases where direct compressive forces cannot be achieved, reinforcing bars positioned across the joint may experience tension under shear loading. This tension can cause joint separation, inducing compressive forces across the interface through the deformation of the bars. Known as the “shear-friction effect”, this mechanism increases shear resistance, as transverse reinforcement and the friction coefficient are enhanced. This concept is frequently employed in grouted longitudinal joints between floor and wall elements, where bond and friction collaborate to facilitate effective force transfer.
4. Post-tensioning is a specialized construction technique primarily used in large-scale structures and in tall buildings made with precast elements. The process consists into casting the concrete element by leaving inside some ducts. After the hardening of concrete, the ducts are filled with tendons. These cables are then tensioned to a specific force and securely grouted, locking the tension in place. This process compresses the concrete elements together, as depicted in Figure 37, enhancing their resistance to tensile and shear forces at the joints. The induced compression significantly improves the stability and durability of the overall structure.

Cohesive connections

Cohesive connections are characterised by material continuity (material interaction) or cohesion between the connecting components. These connections are formed through a material bond between the components, typically occurring at the molecular or atomic level and include methods such as adhesives, welding, or the curing of concrete around reinforcement bars, which create a unified material phase across the joint, enabling effective force transfer. Figure 38 illustrates how forces are transferred through cohesive connections. The resulting bond is often as strong as the materials themselves, providing a durable and frequently permanent connection.

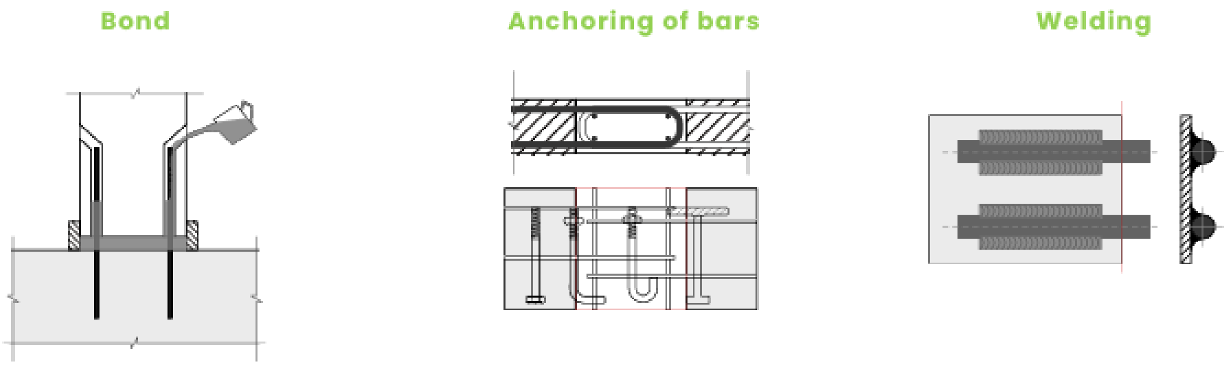

Figure 38 Transfer mechanism based on cohesive connections.

The cohesive connection can be achieved in different ways:

1. The bond mechanism operates through the adhesion and connection between various materials, such as precast concrete elements and cast-in-situ concrete. The efficiency of

bond and shear transfer at the interface depends on several factors, including surface roughness, material strength, cleanliness and the type of treatment applied to the precast surface. This typology of linkage is suitable in general for handling small stresses.

2. The anchorage of reinforcing bars is a crucial aspect of the connections in precast structures. Ribbed bars are used to transfer tensile forces through the interaction between the bar's surface and the surrounding concrete. In this way anchorage capacity must surpass bar's tensile strength, preventing failure mechanisms like concrete splitting or bond failure; to ensure this type of behaviour adequate concrete cover and anchorage length are essential. In situations where space constraints limit the anchorage length, mechanical end anchors such as heads, bends, or hooks are used. These create localized stresses in the concrete, requiring careful design and detailing to avoid brittle failures, as illustrated in Figure 38.
3. Welding offers a durable solution for transferring forces, particularly useful in managing tensile and bending stresses. However, its effectiveness relies on precise execution to maintain the structural performance and reliability of the connections. This technique is widely applied to join steel components within or attached to precast concrete elements, as depicted in Figure 38. Moreover, it enables the direct connection of exposed steel parts, such as overlapping reinforcement bars, to establish a continuous load path and ensure structural integrity at joints.

Form-fitting connections

This category refers to connections that rely on the interlocking shapes or geometries of the components to transfer force effectively. Unlike methods dependent on friction or adhesion, these connections function through direct physical engagement between the parts, with their interlocking design naturally prevent movement or separation under load (Figure 39). In the past was frequent the use of big shear keys to transfer effectively the loads, but this induced huge stress concentration and the need of reinforcement in some zones, so today is preferred to have a higher number of small indentations where the stresses distribute better.

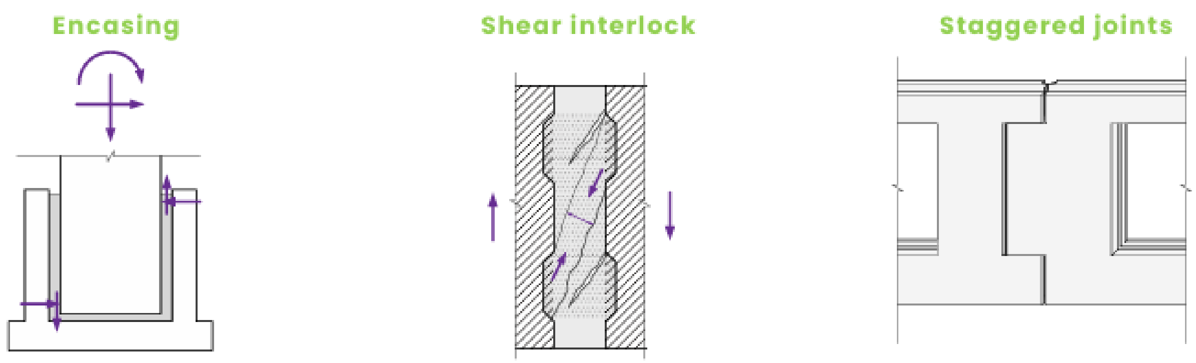

Figure 39 Transfer mechanisms based on shape interlocking.

The most used form-fitting techniques are:

1. Encasing serves as a mechanism for transferring forces in precast construction by inserting one component into another and filling the void between them with a bonding material, such as grout or fine concrete. This technique is primarily used to transfer compression and bending forces. Additionally, encasing enhances shear and torsional resistance by promoting composite action between the connected elements. A common example of encasing is found in Figure 39 where is depicted the connection between a column and a foundation. The bonding material fills the space around the inserted component, securing it firmly in position and facilitating load transfer between the components.
2. Shear interlock operates as a mechanism to transfer shear forces through mechanically interlocked, indented joint surfaces, effectively reducing slip along the joint interface. The compression forces within the mortar counteract the separation of elements under shear loading, as shown in Figure 39.
3. Staggered joints are commonly used in vertical wall joints where shear forces are transferred through compression in the horizontal part of the joint (Figure 39). The horizontal force component generated is counteracted by tie reinforcements positioned above the wall panels. This arrangement facilitates efficient shear force transfer between the elements, with the joint effect of compression and reinforcement promoting balanced load distribution and enhancing the overall stability of the wall system.

2.4 SIMPLIFIED FRAME ANALYSIS

A precast concrete structure is not merely a cast-in-situ system divided into smaller sections for easier transportation and assembly. The transfer of forces through the prefabricated and assembled elements in a precast structure differs significantly from that in a continuous (monolithic) structure, especially near connection points, and so also the analysis of precast concrete structures is different from that of cast in situ ones.

To start a global analysis, can be firstly examined the behaviour of a continuous frame and identified where appropriate connections might be implemented in a precast frame. Initially, a two-dimensional, in-plane simplification can be used, as outlined in Figure 40. Here, structural frame components are absent, with only simple floor units connecting the 2-D in-plane frames.

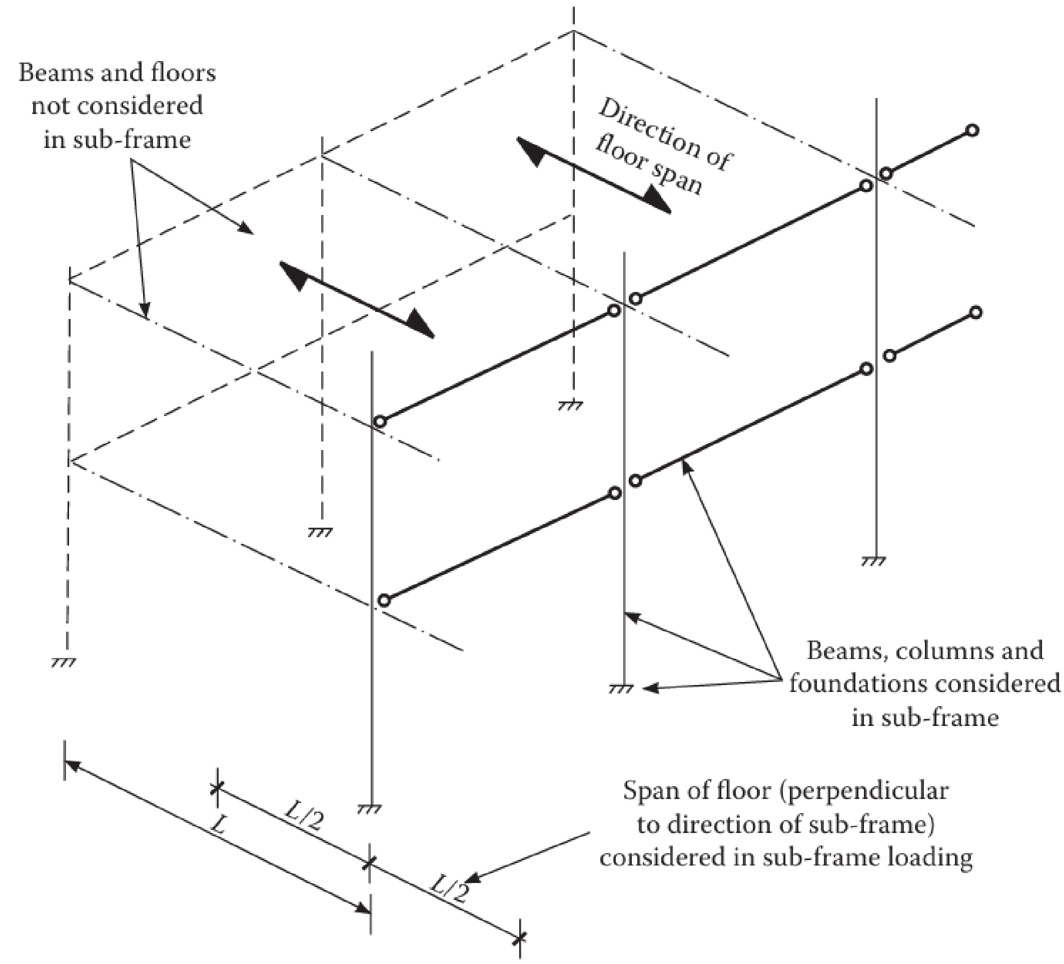

Figure 40 2-D simplification of a 3-D skeletal structure.

It's possible to start a simplified frame analysis by comparing different frames depending on their characteristics (especially stiffness and strength) that will induce different locations of pinned connections [13]. As first stage is possible to consider a simplified three-storey continuous beam-column frame under vertical (gravity) and horizontal (wind) loads where beam-to-column connections possess equal strength and stiffness as the members themselves. This frame is called

Frame F1 and its bending moment and deflected shape are represented in Figure 41. Stability in F1 is achieved through the combined actions of beams, columns, and beam-column connections resisting bending, shear, and axial forces, forming what is known as an "unbraced" frame.

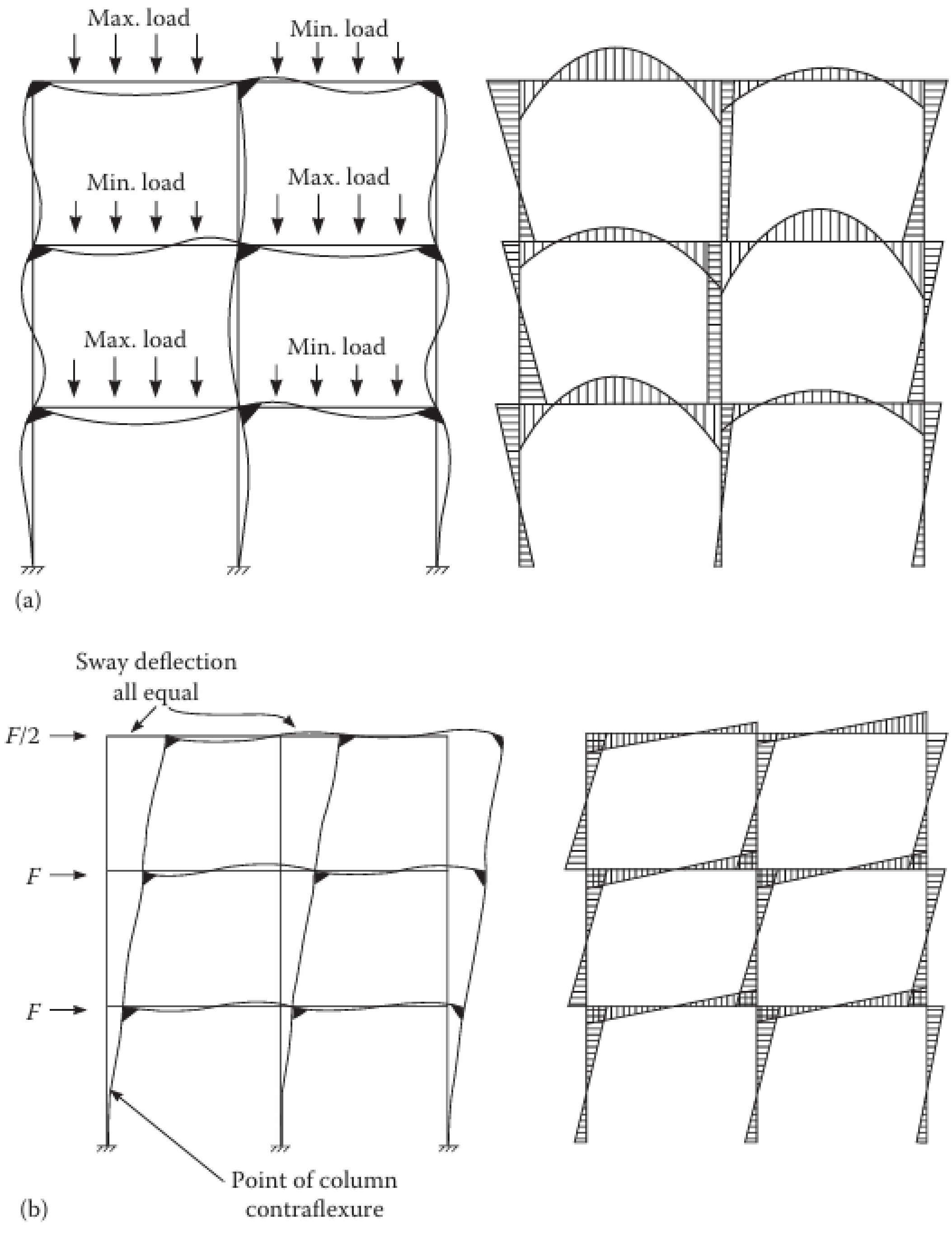

Figure 41 Deformation and bending moment distribution in a continuous structure subjected to (a) gravity loads and (b) horizontal sway load.

F1 exhibits points of zero moment, or contraflexure points, which vary depending on the relative intensity of the vertical and horizontal loads applied. When gravity loads predominate, beam contraflexure points are located near the beam-to-column connections, typically about 0.1 times the beam span, as illustrated in Figure 42. Conversely, when horizontal loads dominate, though this case is less common, contraflexure shifts to mid-span, with the final position for combined loading generally around 0.15 times the beam's span. In columns, however, contraflexure consistently occurs at mid-story height, making it an ideal location for introducing a pinned (notionally low moment capacity) connection between two precast columns.

Figure 42 Beam half-joints at 0.1× span close to points of contraflexure in a continuous beam.

Starting from F1, if the strength and the stiffness of the connection at the end of the beam are reduced to zero, maintaining those of the column and foundation constant, is introduced a new Frame F2 where bending moment and deformed shape are described in Figure 43: it is a pinned jointed unbraced skeletal structure where the stability is given by the columns while beams transfer no moment but only axial and shear forces.

In taller structures exceeding three storeys or approximately 10 meters, the large dimensions required for columns often become impractical and uneconomical, necessitating the use of bracing. Bracing can either extend throughout the full height of the structure, referred to as a "fully braced" frame, or be applied to certain levels only, known as a "partially braced" frame. These distinctions are illustrated in Figure 44. In some cases, bracing may be situated in the upper storeys, provided the columns in the unbraced section below the first floor are stable enough to support horizontal forces and any second-order moments caused by slenderness.

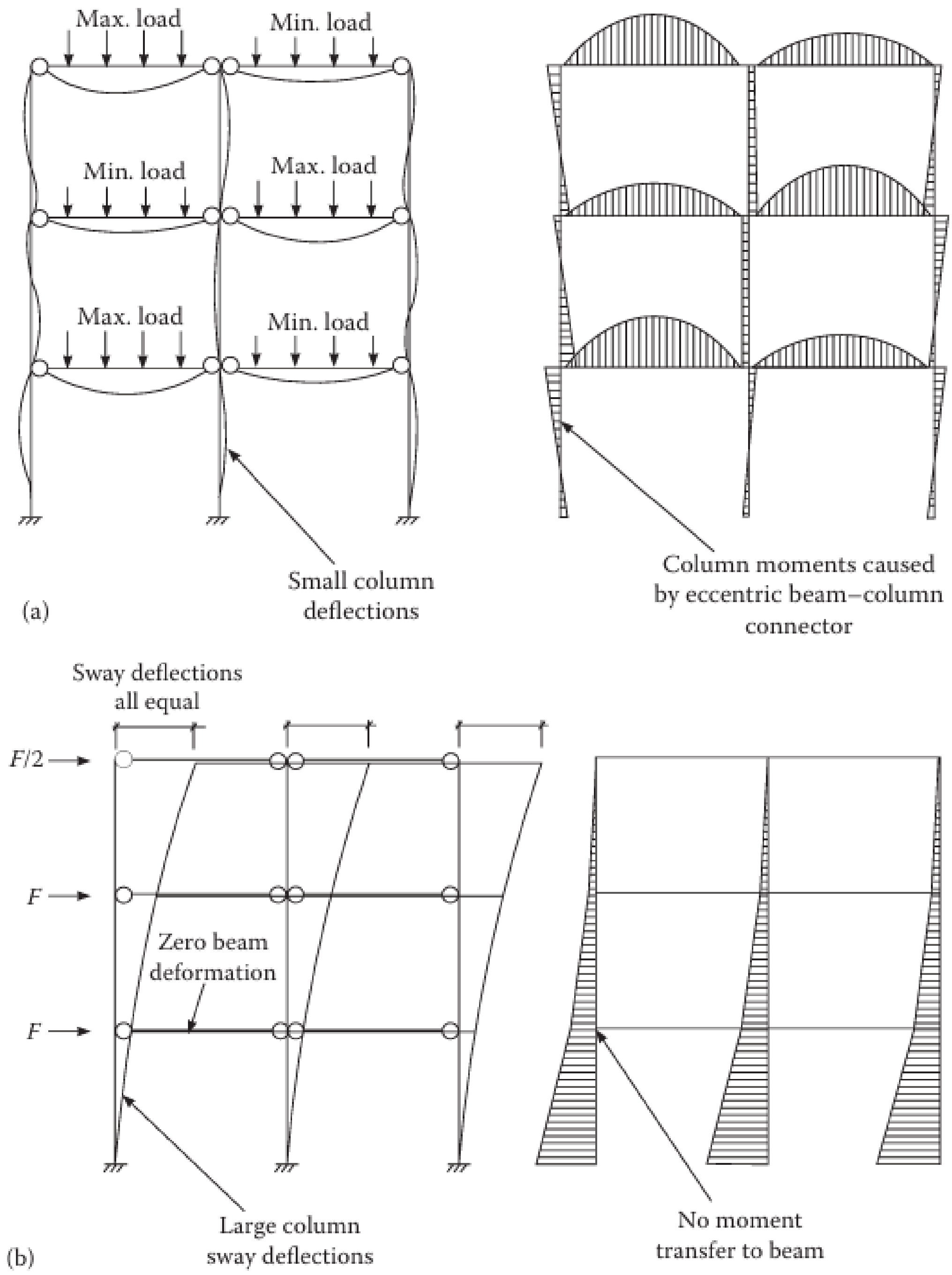

Figure 43 Deformation and bending moment distribution in a pinned jointed structure subjected to (a) gravity loads and (b) horizontal sway load.

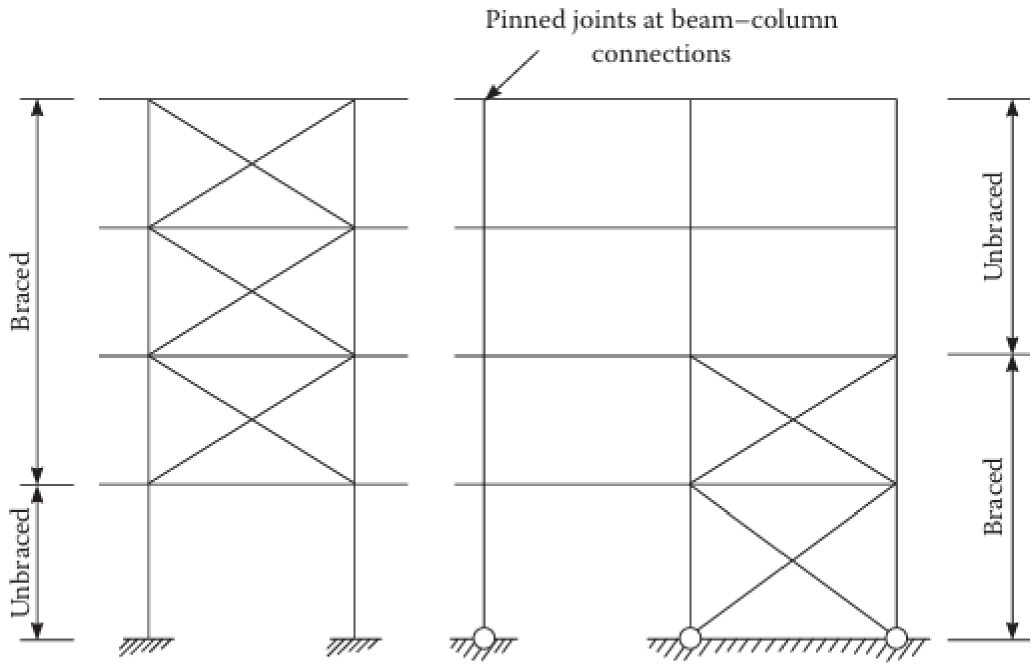

Figure 44 Partially braced structures.

Pinned connection may form at other locations depending on the flexural stiffness of the members. In general, when the flexural stiffness at the lower end of a column exceeds that at its upper end, the point of contraflexure will be located closer to the lower (stiffer) portion of the column. In particular, restarting from frame F1 and maintaining untouched the beam and the beam-column connection, if the stiffness and strength at the lower end are reduced to zero the behaviour of this modified frame, designated as F3, results in specific moments and deflections, illustrated in Figure 45a. The stability of this frame relies on the inverted U-shaped portal action; however, this is not so practical for structures such as parking with long span beams and low storey height columns. In these cases is preferred a most effective solution made by prefabricated L-frames (Figure 45b) where the connections can be either pinned or fixed, but in general the latter ones are chosen to ensure safety and immediate stability during construction.

Another possible variation is the so-called H-frame. It is obtained starting from F1 and putting pinned connections exactly at points of column contraflexure. In this way the connections between frames are made at mid-storey height, base connections are fixed and all the others must possess flexural strength, so the overall the structural behaviour is near to that one of a continuous frame as shown in Figure 46. It's important to notice that although the connection is theoretically treated as pinned, a small amount of moment transfer is practically unavoidable. Consequently, H-frame connections are equipped with a finite moment capacity, improving both the safety and stability of H-frames, which by their nature tend to be massive.

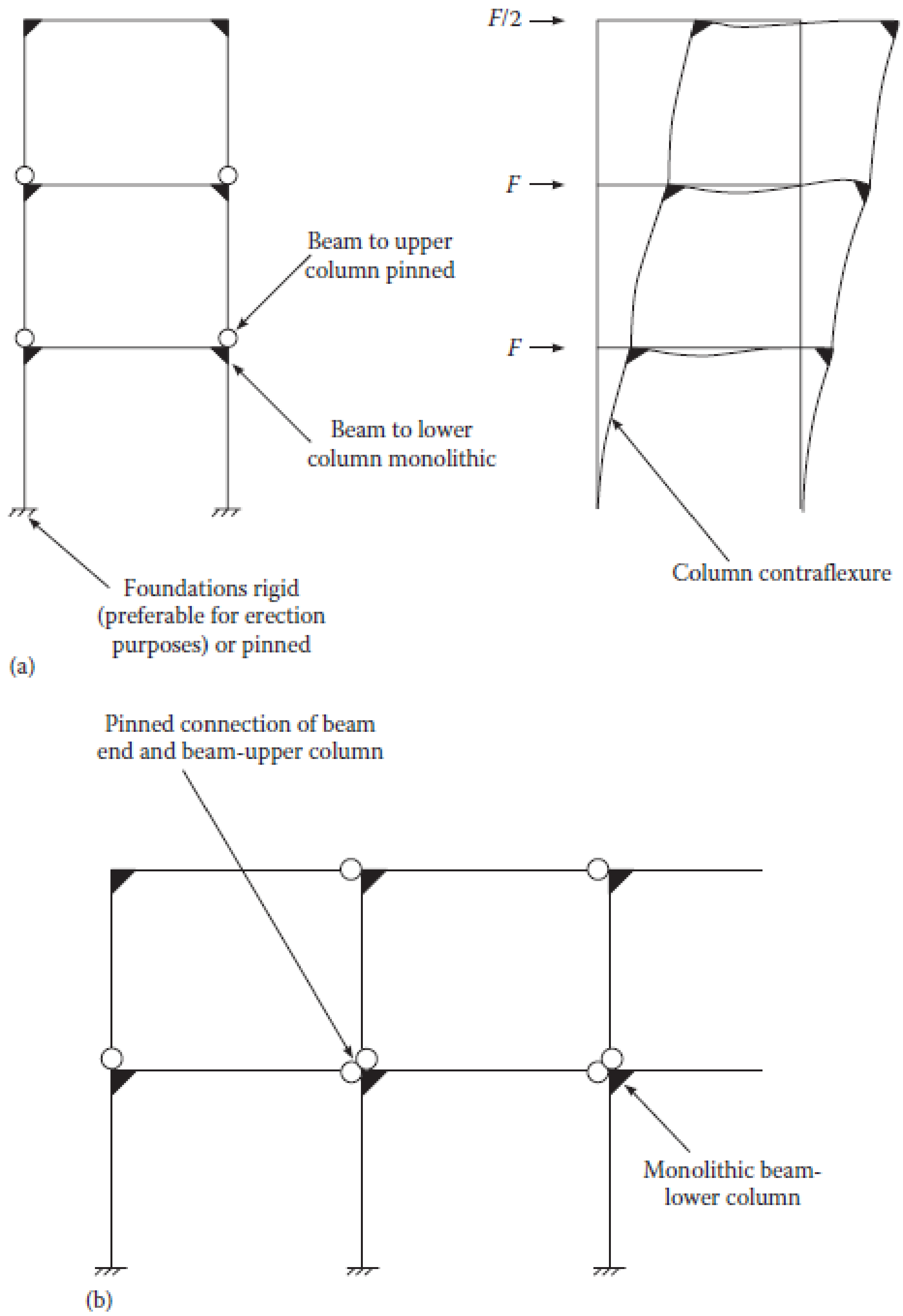

Figure 45 Structural systems for (a) portal U-frames and (b) portal L-frames.

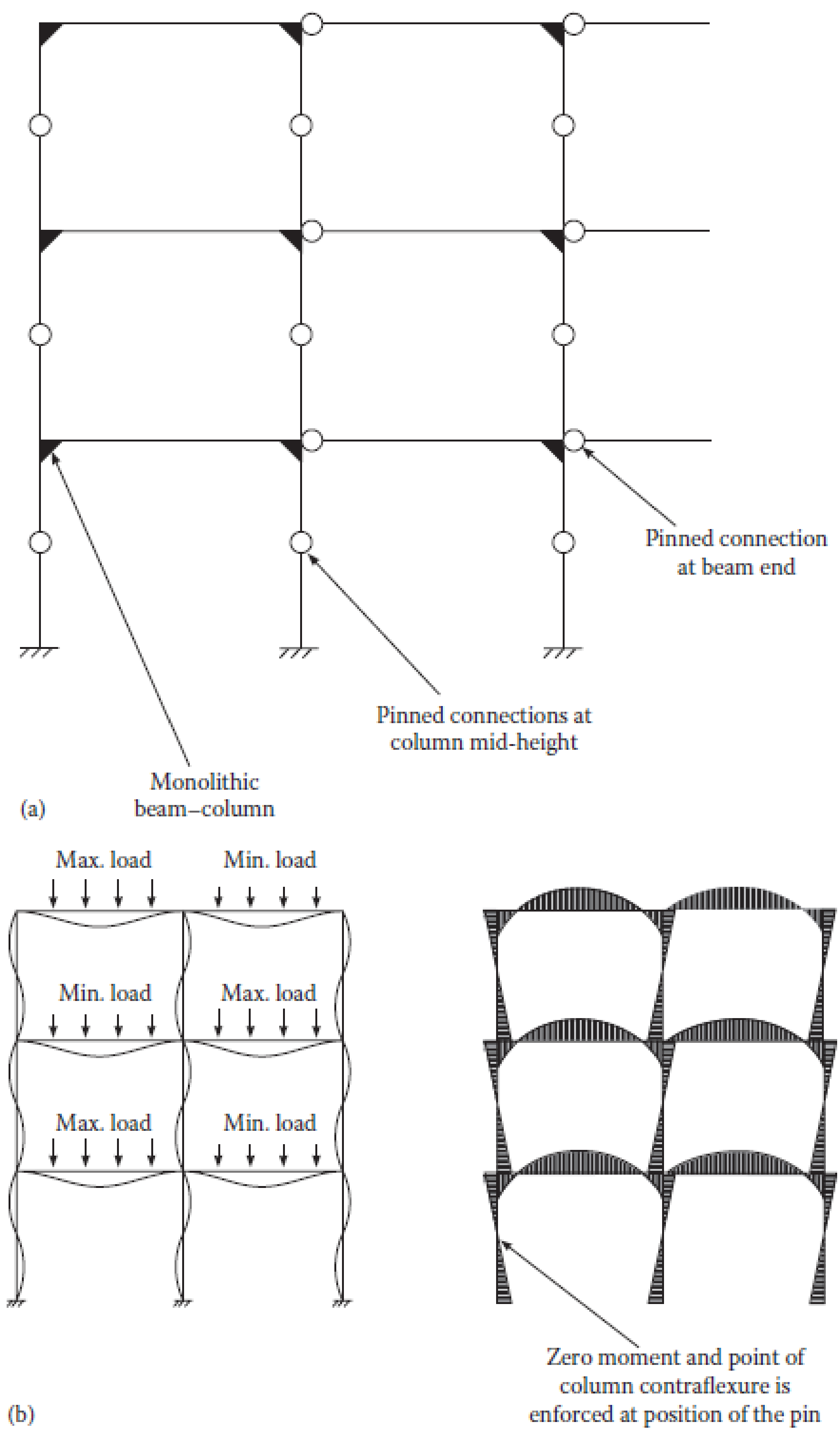

Figure 46 H-frames: (a) structural system, (b) deformation and bending moments.

2.4.1 Sub-structuring methods

The purpose of structural analysis is to compute bending moments, shear forces and axial forces over a structure. Two-dimensional monolithic plane frames are typically analysed using rigorous elastic methods, which can be applied manually or through computer programs. Moment redistribution may be incorporated into the analysis when suitable. However, in many cases, it's only necessary to calculate the forces and moments for a specific beam or column. To simplify this process, building codes often allow for sub-structuring techniques, which are practical for determining these values [4].

Sub-structuring involves dividing the entire structure into smaller substructures, analyse each one independently and finally reassemble them by satisfying compatibility and equilibrium conditions at joints. It is an ideal method for precast structures because it mirrors the real construction process: analyse individually the elements and then connect them. When the structure is relatively uniform, meaning spans and loads differ by no more than 15%, the results from sub-structuring typically align with those from full-frame analysis, achieving an accuracy of 90% - 95%.

The method can be used also for common cast in situ frames with rigid connections, an example can be seen in Figure 47. Nevertheless, this section will be focused in its use for precast structures generally characterised by pinned column-beam connections, where no moment redistribution is allowed.

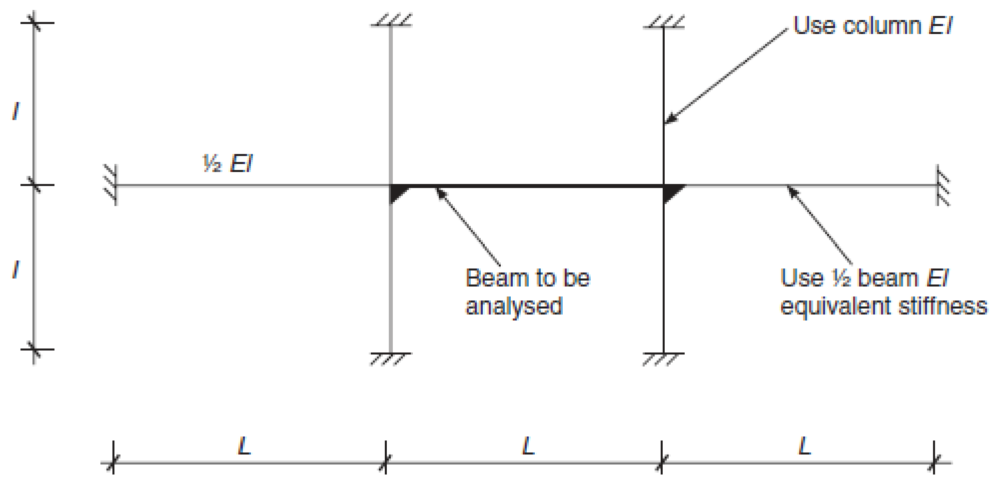

Figure 47 Sub-structuring method for internal beam in a continuous cast in situ frame.

Figure 48 and Figure 49 show the sub-frame considered for the analysis of internal beam and upper and ground floor column where all beam-column connections are pinned and base connections are fixed. An elastic method is used to determine internal actions and deflections, while a plastic section is used for the design of the elements. Horizontal wind loads are not considered in subframes because bring very small bending moment into the column respect to those derived from vertical loads.

The critical loading combinations to be considered are:

1. All spans loaded with the maximum ultimate load $\omega_{max} = \gamma_f\, g_k + \gamma_f\, q_k$ and;
2. Alternate spans loaded with the maximum ultimate load, ω_{max} , on one span and the minimum $\omega_{min} = 1.0g_k$ on the adjacent.

Where g_k and q_k are characteristic dead and variable uniformly distributed loads and γ_f are the associated partial safety factors for loads.

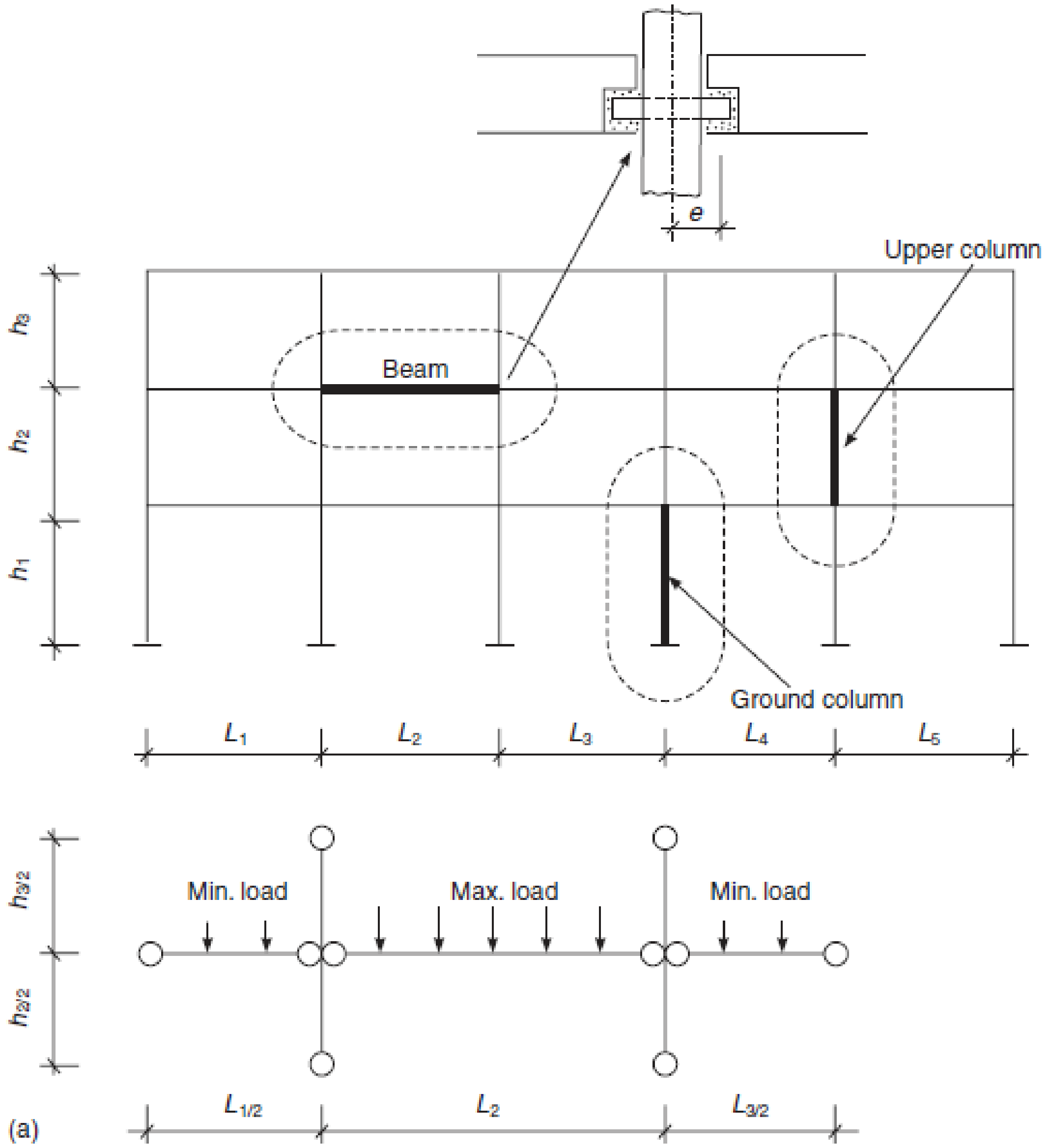

Figure 48 Sub-structuring methods for internal beam and columns in a pinned-jointed frame.

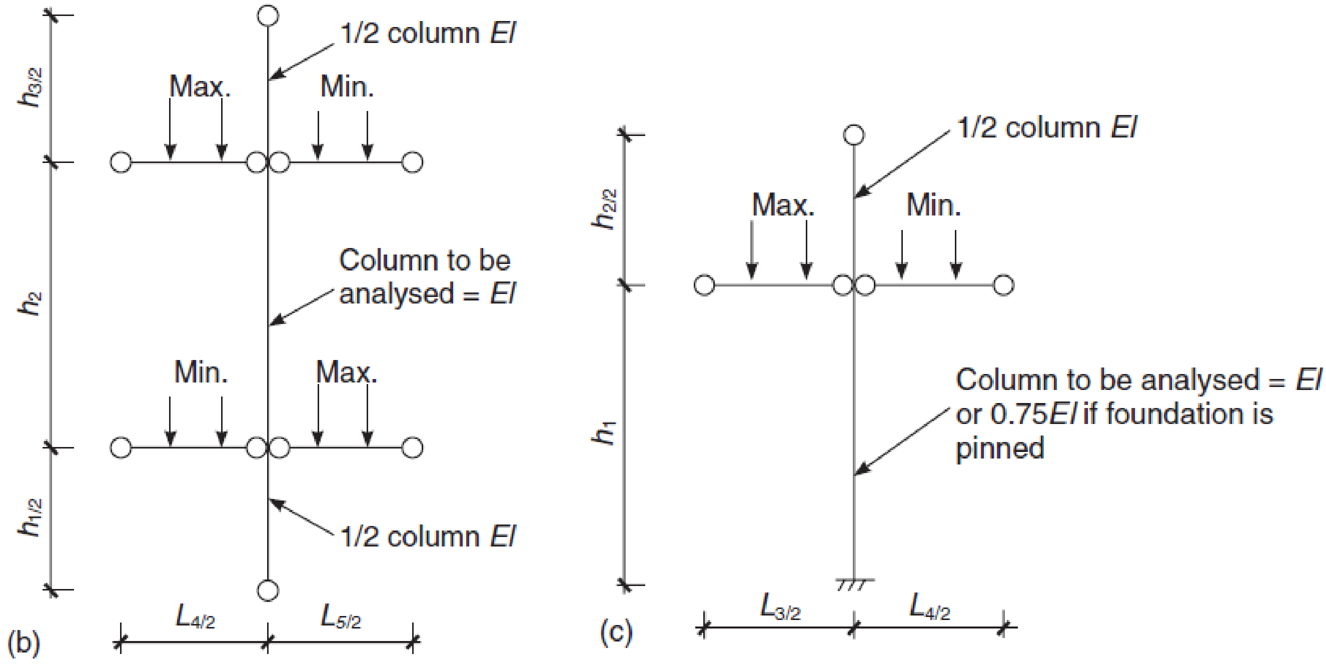

Figure 49 Sub-structuring methods for internal beam and columns in a pinned-jointed frame.

Beam subframe

The subframe includes a beam with a span of L_2 that needs to be designed, along with half of the adjacent beams spanning L_1 and L_3. The eccentricity of the beam end reaction relative to the centroidal axis of the column is denoted as *e*. Alternate pattern loading is applied. The height of the column above and below the beam does not influence the beam's performance. It is assumed that the column has a uniform cross section and constant flexural stiffness.

The maximum moment in the beam is

$$M_1 = \frac{\omega_{max}(L_2 - 2e)^2}{8} \tag{1}$$

The beam end reaction is

$$R_1 = \frac{\omega_{max}\, L_2}{2} \tag{2}$$

End reactions in the adjacent beams are

$$R_1 = \frac{\omega_{min}\, L_1}{2} \;\; and\; R_3 = \frac{\omega_{min}\, L_3}{2} \tag{3}$$

The resulting maximum bending moment in the column in given by:

$$M_{col} = \frac{(R_2 - R_1)e\, h_2}{(h_2 + \; h_3)} \tag{4}$$

assuming that $R_1 < R_3$ and $h_3 > h_2$.

The final bending moment are given in Figure 50.

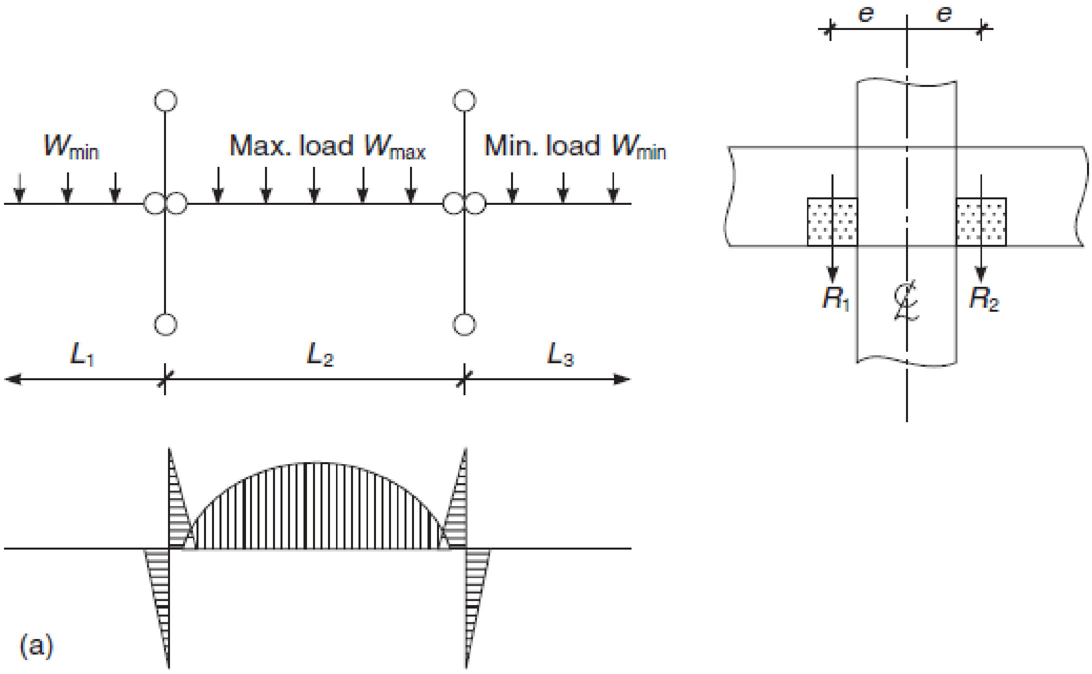

Figure 50 Bending moments in a pinned-jointed frame for internal beams.

Upper floor column subframe

The subframe consists on the column to be designed, which has a height h_2 (measured as the distance between the centres of beam bearings), along with half of the adjacent columns of heights h_1 and h_3. Since the column is continuous, the cross-sectional properties and flexural stiffness (EI) for each segment of the column are accounted for, as illustrated in the figure 48b. The beams are subjected to pattern loads, with spans of $L_4/2$ and $L_5/2$. Additionally, the eccentricity of the end reaction forces from each beam relative to the column's centroidal axis is denoted by e_4 and e_5, respectively.

The resulting moment at the upper end of the designed column is given by

$$M_{col,upper} = (R_4 e_4 - R_5 e_5)\frac{\frac{EI_2}{h_2}}{\frac{EI_2}{h_2} + \frac{EI_3}{h_3}} \tag{5}$$

The resulting moment at the lower end of the designed column is given by

$$M_{col,lower} = (R_4 e_4 - R_5 e_5)\frac{\frac{EI_2}{h_2}}{\frac{EI_2}{h_2} + \frac{EI_1}{h_1}} \tag{6}$$

Where $R_4 = \frac{\omega_{max} L_4}{2}$ and $R_5 = \frac{\omega_{max} L_5}{2}$.

The final moments are given in Figure 51.

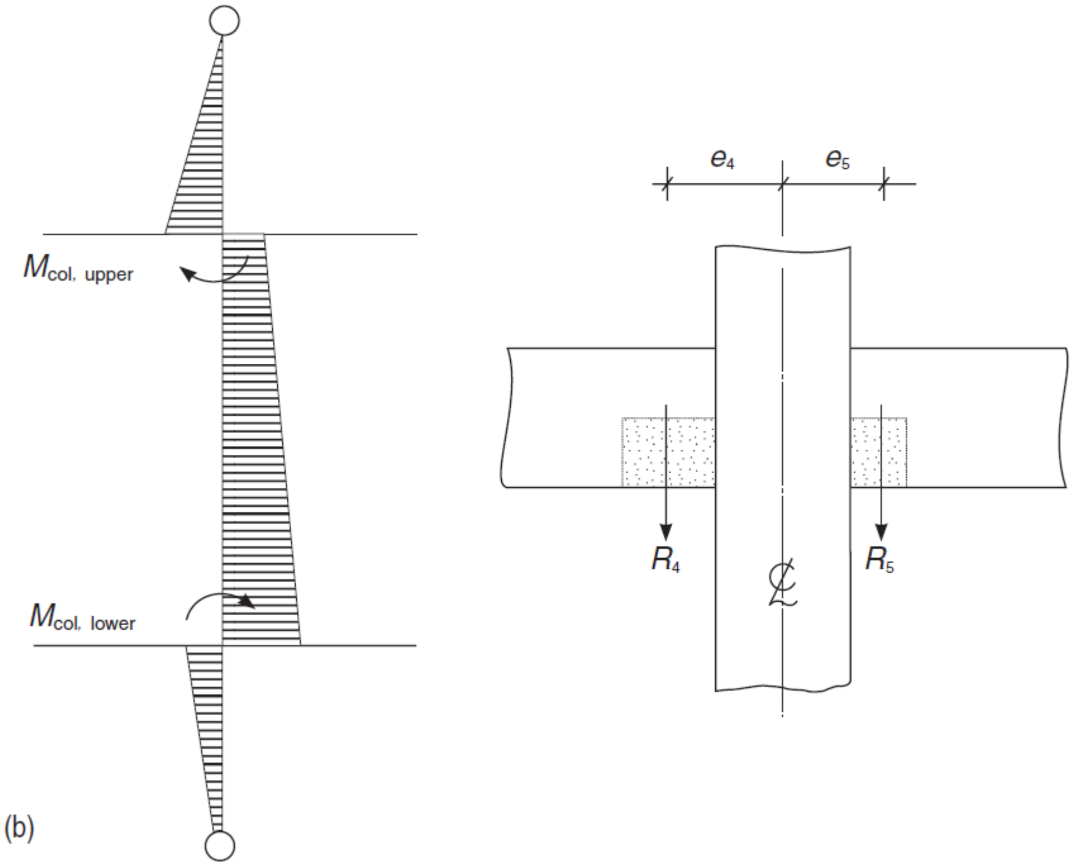

Figure 51 Bending moments in a pinned-jointed frame for upper floor columns.

Ground floor column subframe

The subframe includes the column to be designed, which has a height measured from the centre of the first-floor beam bearing to 50 mm below the top of the foundation, denoted as h_1, and half the adjacent column of height h_2 (Figure 49c). All other aspects remain consistent with previous specifications. The evaluation of bending moments at upper and lower ends in the column depend on the type of connection at the level of the foundation. In cases where the foundation is rigid and capable of resisting moments, the moment at the upper end of the designed column can be determined using Equation (5) with the appropriate variables. Additionally, the carry-over moment at the lower end is equal to 50% of the moment at the upper end.

If instead the foundation is pinned, the lower moment is zero and the upper moment is given by:

$$M_{col,upper} = (R_4 e_4 - R_5 e_5) \frac{0.75 \frac{EI_1}{h_1}}{0.75 \frac{EI_1}{h_1} + \frac{EI_2}{h_2}} \quad (7)$$

Figure 52 shows the final moments in the ground floor column subframe for both cases.

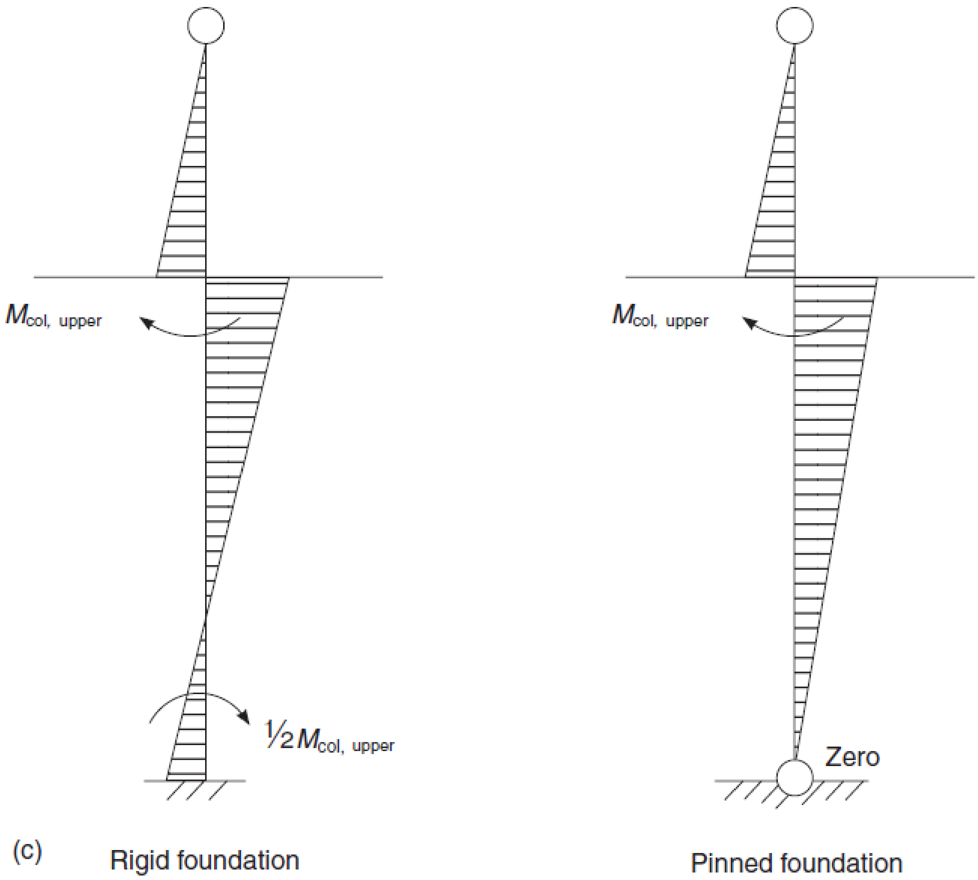

Figure 52 Bending moments in a pinned-jointed frame for ground floor columns.

2.4.2 Connection design

Connections are an essential component in the design and construction of precast concrete systems. They significantly influence the type of structural frame utilized, its limitations, and the efficiency of erection procedures. For load-bearing wall systems, connection rigidity can be remarkably low, sometimes just 1/100 of the stiffness of the wall panels and so as a consequence deformability is high and can be 10 times greater than that of the panel.

While the terms "connection" and "joint" are frequently used interchangeably, this section clearly distinguishes between them, by referring to definitions in [13]:

- Connection: is the total construction between two (or more) connected components, it includes a part of the precast component itself and may comprise several joints.
- Joint: is the part of a connection at individual boundaries between two elements (the elements can be precast components, in situ concrete, mortar bedding, mastic sealant, etc.)

For example, in a typical beam-column assembly (illustrated in Figure 53), multiple joints come into play: a bearing joint between the beam and the column corbel, a shear joint at the dowel-to-angle interface, a bolted joint connecting the angle to the column. Once these joints are consolidated with in-situ mortar or grout, they collectively form a connection whose overall performance depends on both the precast elements and all interfacing joints.

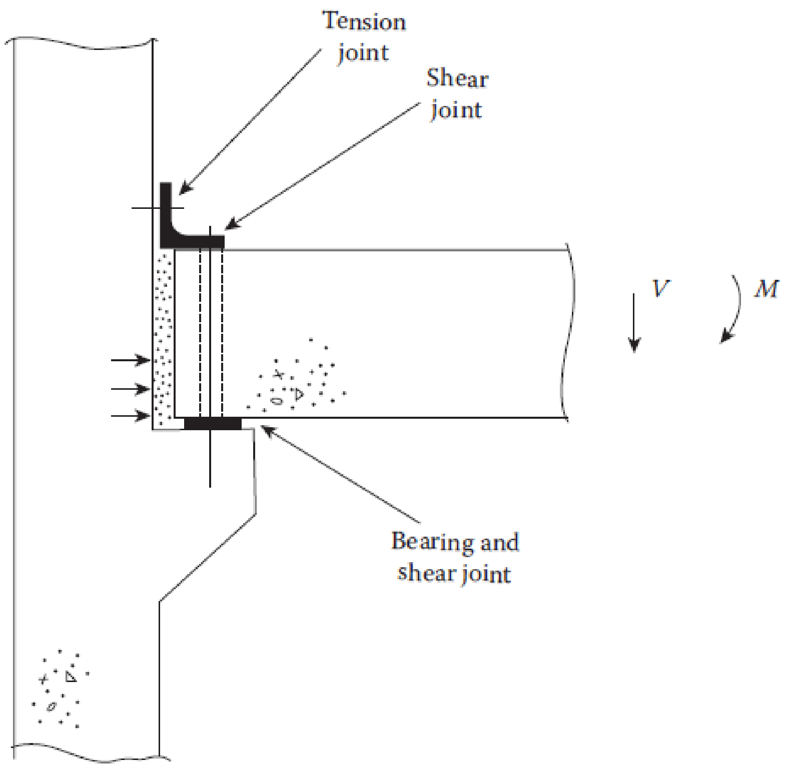

Figure 53 Moment and shear transfer at a bearing corbel.

From a structural standpoint, joints must be designed to effectively transfer various types of forces, including compression, shear, tension, bending, and occasionally torsion. When concrete alone is not able to carry the applied stresses, steel inserts or other embedded materials can be incorporated to enhance capacity. However, regions with localized stress concentrations will arise, generally near inserts or geometric irregularities, and require detailed evaluation to confirm the structural strength of each joint.

Other important factors to be considered in connection design are manufacturing tolerances, erection methods and workmanship, which play a critical role in overall performance. To guarantee the integrity of the structure, it is vital to understand how forces are transmitted through each connection and confirm that all joints can adequately handle their loads.

Making a comparison between cast in situ concrete and precast construction, in the latter one are present some additional forces to be taken into account, these include:

- Frictional forces arising from differential movements (for example shrinkage effects),
- Pretensioning stresses in both concrete and reinforcement steel,
- Stresses from handling, self-weight during lifting, transportation, and assembly operations.

For instance, Figure 54 illustrates the interaction between a reinforced concrete column and corbel supporting a pretensioned concrete beam. This example highlights the complex interaction between forces and the need for specific analysis for the joint design to ensure structural reliability and safety.

The figure shows that there are 10 different force vectors in this connection as follows:

- A: diagonal compression strut in corbel
- B: horizontal component reaction to force A
- C: vertical component reaction to force A
- D: internal diagonal resultant to forces B and C
- E: diagonal compression strut in beam
- F & G: horizontal component reaction to force E
- H: tension field reaction to forces E and F
- J: horizontal friction force caused by the relative movement of beam and corbel
- K: horizontal membrane reaction to beam rotation due to eccentric prestressing

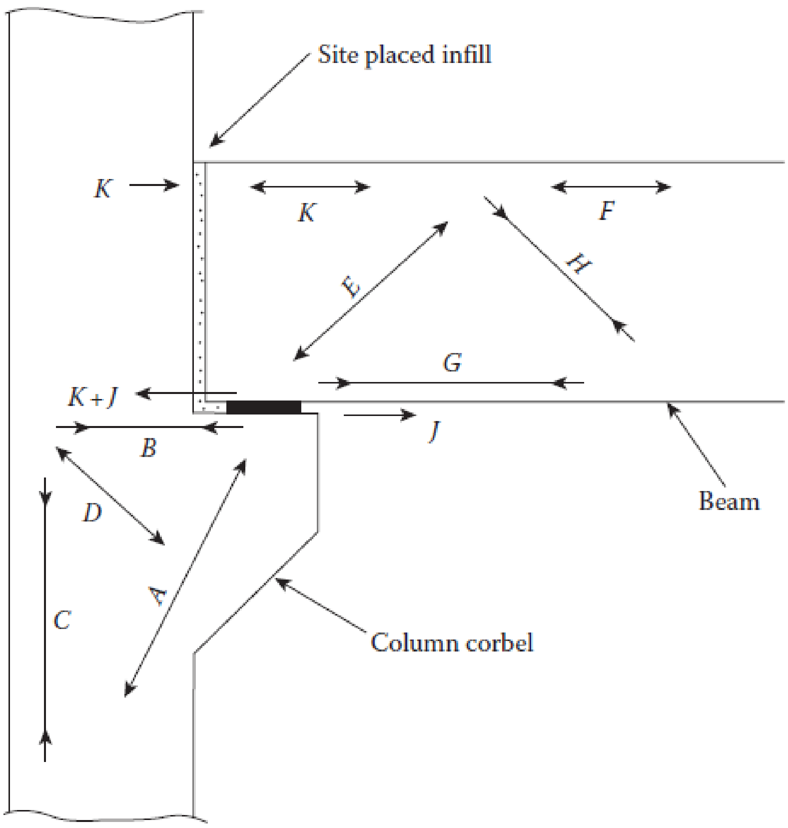

Figure 54 Force paths in beam to column corbel connection.

To obtain the various structural systems described in the previous chapters it's often necessary to design and use rigid, pinned connections or combinations of them. In precast systems generally designers prefer bolted or welded connections respect to monolithic rigid ones because they enable quicker assembly processes and greater reliability. On the other hand, achieving monolithic rigid behaviour is possible only during the casting phase. This concerns some disadvantages linked to cast in place concrete such as:

- Sensitivity to weather conditions (particularly in colder climates),
- Slower strength development,
- Larger cross-sectional dimensions,
- Dependence on construction quality.

In some specific locations, such as foundations, where space constraints are less critical, rigid connections are more feasible.

Figure 55 and Table 6 Types of connections in precast frames.are useful to gain a general overview of the various types of connections relating their location and basic construction into a generic structure.

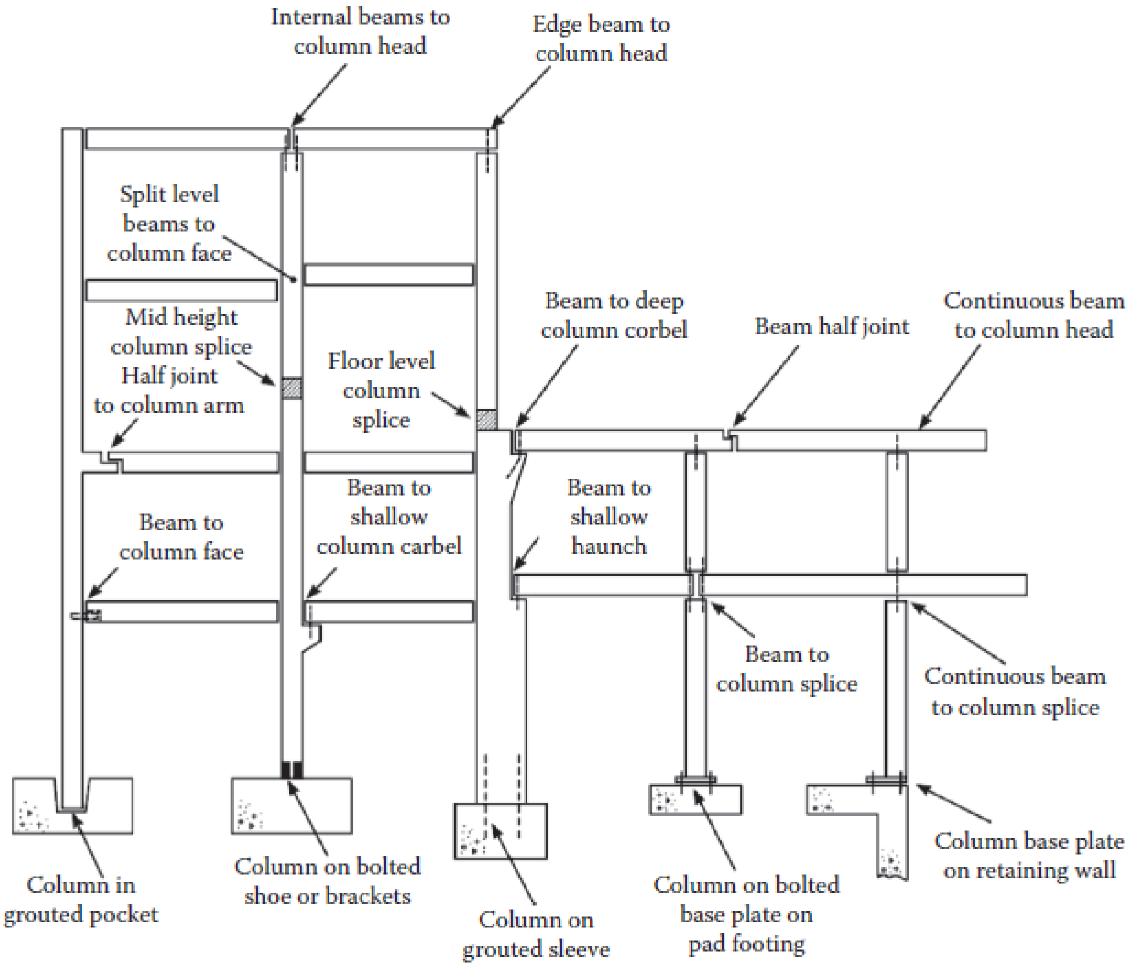

Figure 55 Types of connections in a precast structure.

Connection type	Location in Figure	Classification	Method of jointing
Beam – column head	1	Pinned	Dowel
Beam – column head	2	Rigid	Dowel plus continuity top steel
Rafter – column head	3	Pinned	Dowel
Rafter – column head	4	Rigid	Bolts (couple)
Column splice	5	Pinned	Bolts/dowel
		Rigid	Bars in grouted sleeve (couple)
			Threaded couplers
			Steel shoes
Beam – column face	6	Pinned	Bolts
			Welded plates
			Notched plates
			Dowels
Beam – column corbel	7	Pinned	Dowel
Beam – column corbel	8	Rigid	Dowel plus continuity top steel
Beam – beam	9	Pinned	Bolts
			Dowels
Slab – beam	10	Pinned	Tie bars
Slab – wall	11	Pinned	Tie bars
Column – foundation	12	Pinned	Bolts
Column – cast *in situ* beam or retaining wall	13	Pinned or rigid[a]	Bolts Rebars in grouted sleeve

[a] Depending on the design of the cast *in situ* substructure.

Table 6 Types of connections in precast frames.

In theory, no connection is ever perfectly rigid or perfectly pinned, all behave in a semi-rigid way, especially after flexural cracking takes place. To assess the performance of these connections, engineers can use the beam-line analysis method, as illustrated in Figure 56. Initially designed for steel structures in the 1930s, this approach has been successfully adapted and validated for precast concrete applications over time.

The moment-rotation (M-θ) diagram is developed based on theoretical extremes as follows.

For a fully rigid support, the hogging moment of resistance is expressed as:

$$M_{rd} > \frac{\omega L^2}{12} \tag{8}$$

For a pinned beam subjected to a uniformly distributed load (UDL) of intensity ω, the corresponding rotation is given by:

$$\theta = \frac{\omega L^3}{24EI} \tag{9}$$

The gradient of the beam line can be represented as:

$$\frac{M}{\theta} = \frac{2EI}{L} \tag{10}$$

Using this information, the M-θ diagram is constructed with the following plot lines:

- Plot 1 symbolizes an ideal rigid or monolithic connection.
- Plot 2 illustrates a pinned connection.
- Plots 3, 4, and 5 depict typical semi-rigid behaviours commonly noted in precast structural systems.

When the M-θ plot (for example Plot 5) does not intersect the beam line, the connection is classified as lacking adequate ductility and should be treated as pinned for design considerations. If the plot (for example Plot 3) falls within the shaded region but extends beyond its limits upon failure, the connection's behaviour closely resembles that of a monolithic structure, with a deviation of less than 5%. In such cases, it can be regarded as rigid for design purposes.

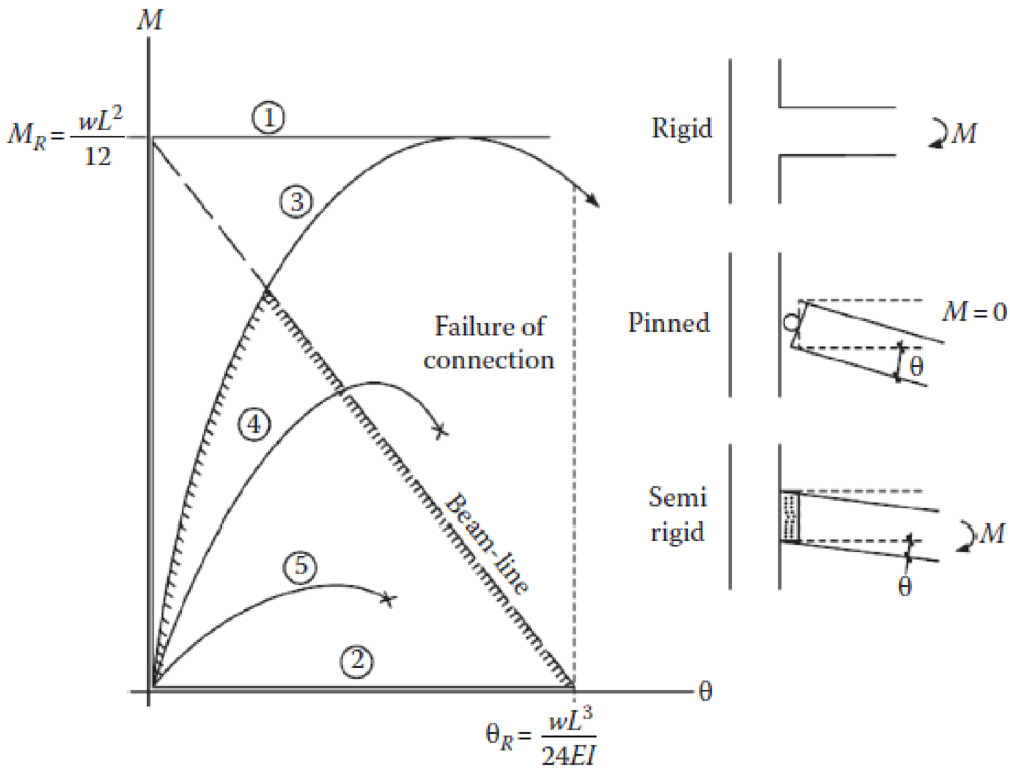

Figure 56 Moment - Rotation diagram and characteristics.

2.4.3 Stabilising methods

Structural stability and safety are fundamental aspects to take into consideration during the erection of precast concrete frames. The complete structural system achieves full stabilisation only after all connections are properly completed. This process may require several hours or days, depending on the kind of connection, and during that time temporary bracings or support systems might be necessary to ensure stability.

A stabilising system is characterised by two essential components (Figure 57):

- A horizontal system, commonly referred as “floor diaphragm” that distribute the load in horizontal direction across the structure, acting as a deep beam subjected to bending moment and shear forces.
- A vertical system, responsible for transferring reactions from the diaphragm down to the foundations. This can be achieved through shear walls, cores or braced frames.

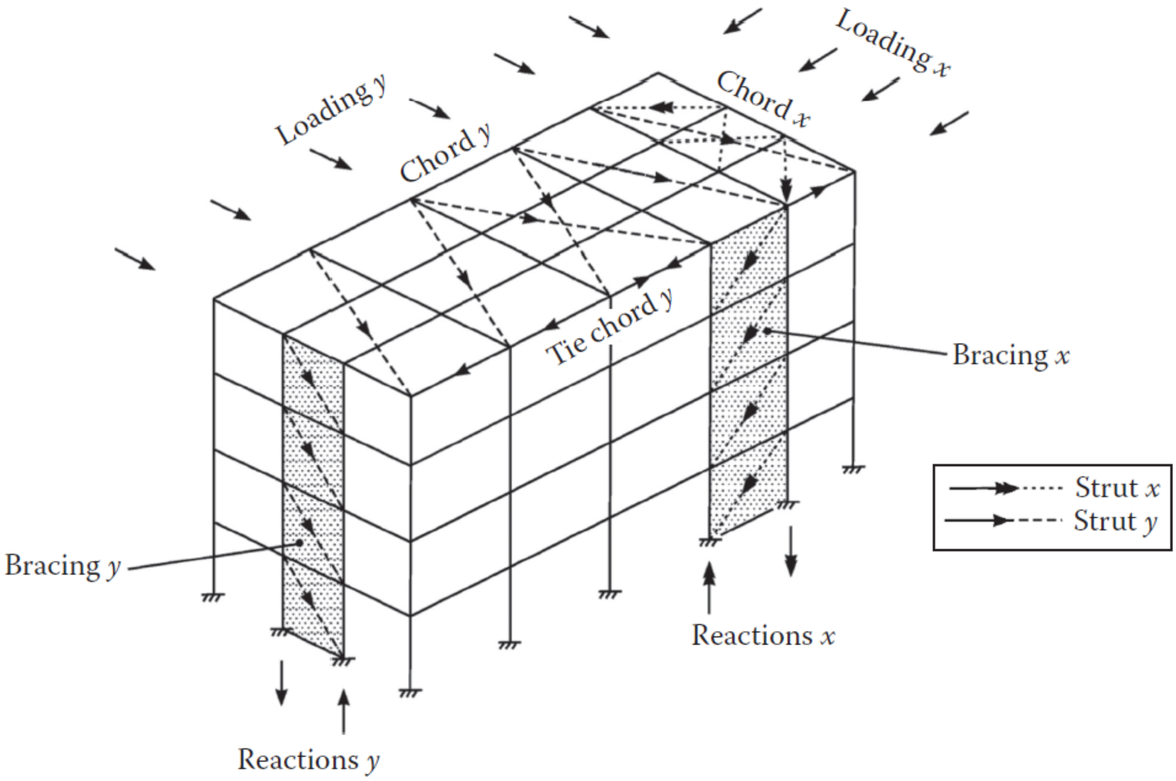

Figure 57 Stabilising systems in braced frames.

The design is a three-stage approach:

1. The floor diaphragm is modelled as long deep beam supported by multiple vertical elements that can be shear walls, cores and bracings. (Figure 58a)
2. If there are only two supports, the system is statically determinate, and the bending moment $M_{ed,h}$ and shear force $V_{ed,h}$ can be calculated directly. With more than two supports, the system becomes statically indeterminate. The support reactions must be determined using analysis techniques that consider relative stiffness and position of each support and the applied horizontal load distribution.
3. The required area of reinforcement needed to carry these forces is computed using the following formulations.
   For bending moment:

$$A_{sh} = \frac{M_{ed,h}\gamma_m}{0.8Bf_{yk}} \tag{11}$$

Where 0.8B is the assumed level arm, γ_m / f_{yk} is the design yield stress of reinforcement with $\gamma_m = 1.15$

For shear forces:

$$A_{shd} = \frac{V_{ed,h}\gamma_m}{0.6\mu f_{yk}} \tag{12}$$

Where μ is the coefficient of friction depending on the typology of surfaces considered.

The tie steel A_{shd} everywhere moments occur. Then where shear forces are greater than a threshold value some additional reinforcement must be placed.

Diaphragms can be reinforced in various ways. In Figure 58b, a reinforced cast-in-place topping layer is used to transfer all horizontal forces to the vertical structural system, with the precast floor serving only to prevent the topping from buckling. In contrast, Figure 58c illustrates a scenario without a cast-in-place topping where the perimeter and internal tie steel resist chord forces generated by horizontal moments.

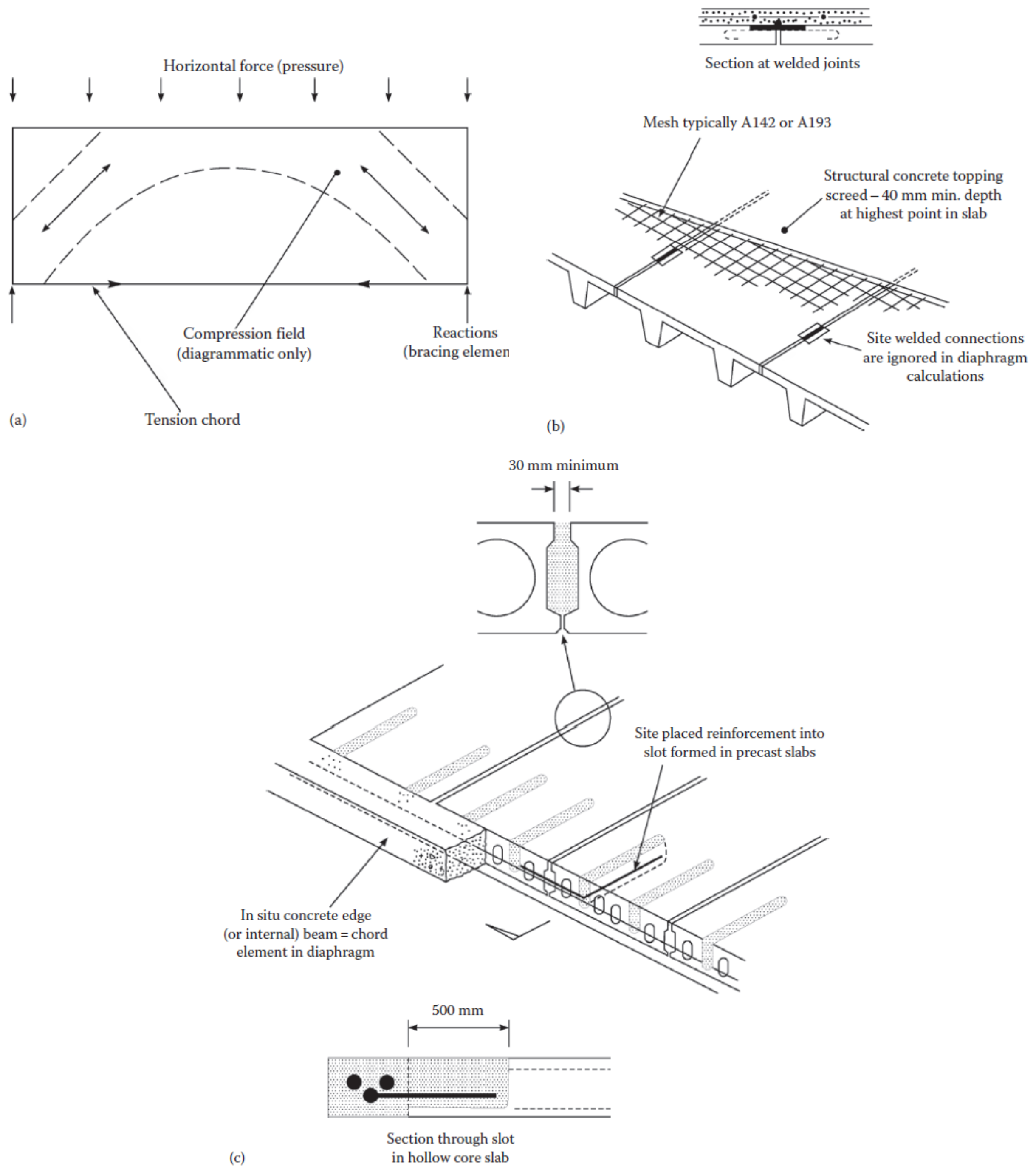

Figure 58 Diaphragm floor action. (a) Deep beam analogy. (b) Reinforced structural topping in double tee floors. (c) Perimeter reinforcement in hollow core floors.

Vertical stabilizing systems are governed by the functional demands of the structural framework, whether it is skeletal, wall-based, or a portal frame. Column effective lengths will depend on the typology and direction of the bracing. Structures can typically be categorized into three main types:

1. Unbraced Frame: As illustrated in Figure 59, the resistance to horizontal forces is achieved through moment-resisting frame action, cantilever behaviour of columns, or cantilever action deep columns (cantilever wind post).

2. Braced Frame: Shown in Figure 60, horizontal force resistance is provided by various mechanisms, such as cantilever action of walls or cores, in-plane panel action from shear walls or cores, infill walls, cross bracing, and related systems.
3. Partially Braced Frame: Depicted in Figure 59, this type combines characteristics of both unbraced and braced frames, integrating elements from the first two categories for stabilization.

The choice of stabilizing system can vary in different directions based on the floor plan layout and the availability of shear walls or structural cores.

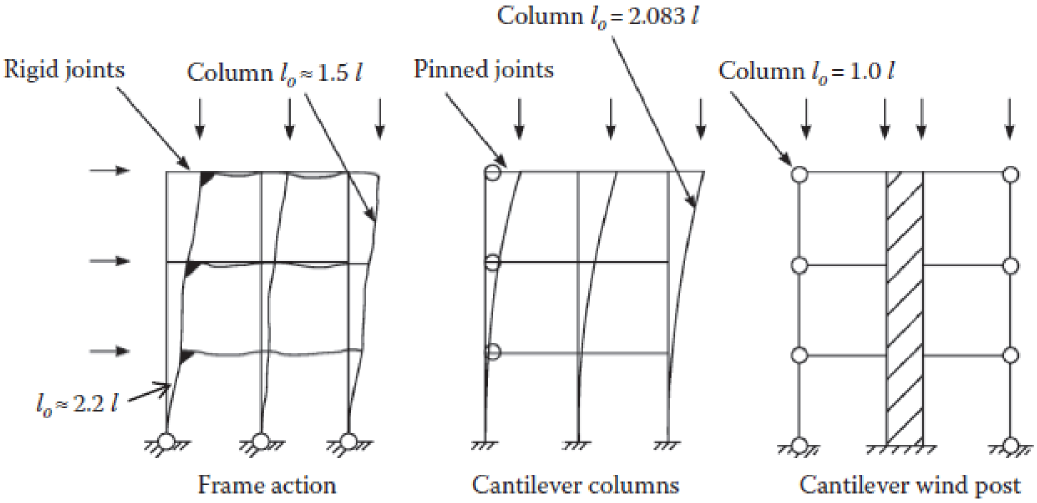

Figure 59 Alternative sway mechanisms and resulting column effective length factors.

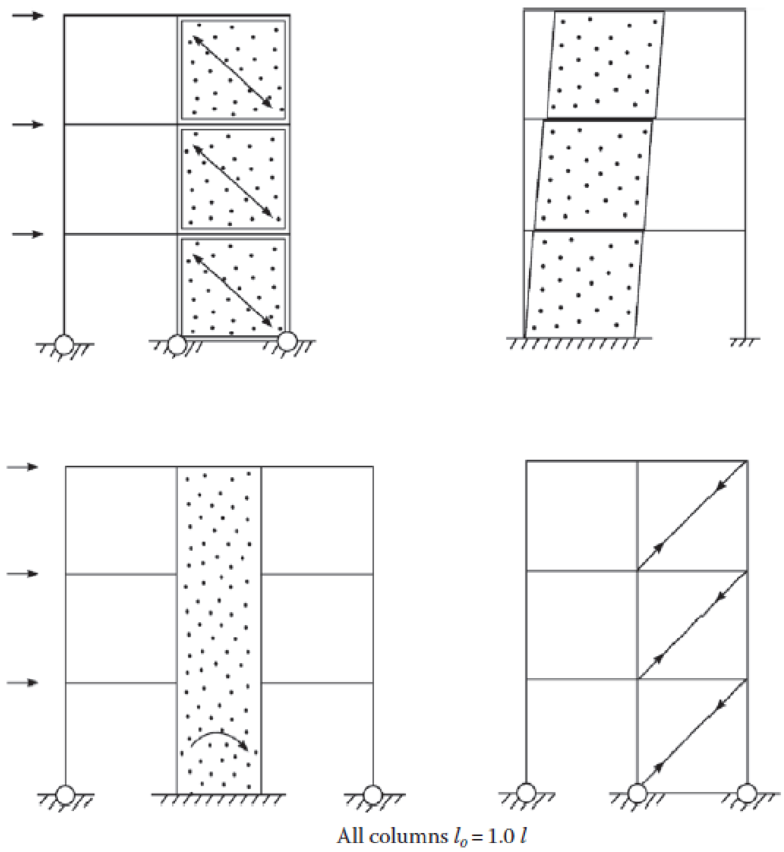

Figure 60 Alternative full-height bracing mechanisms and resulting column effective length factors.

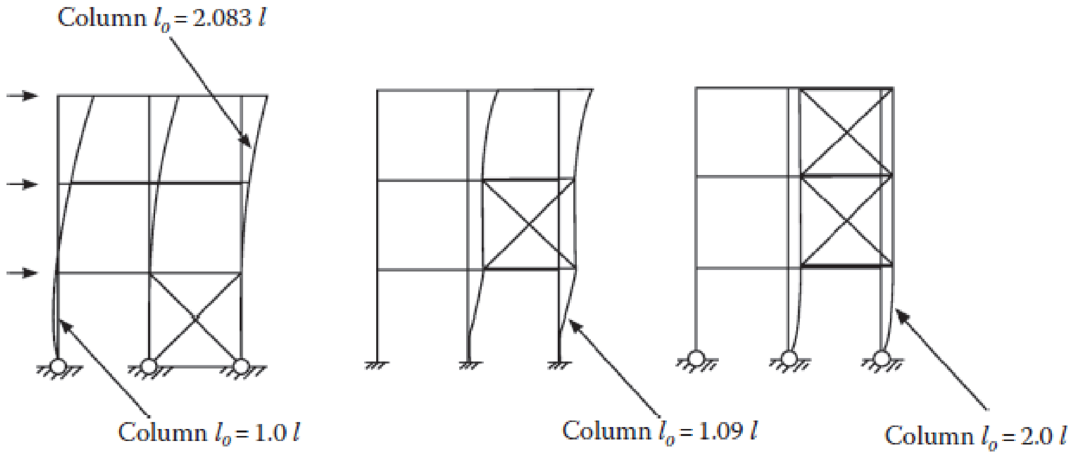

Figure 59 Alternative partial height bracing mechanisms and resulting column effective length factors.

2.5 MATERIALS

2.5.1 Concrete for precast production and main properties

Precast concrete is widely acknowledged for its highest quality, particularly regarding strength and durability. The use of materials and aggregates in the production subjected to stringent quality controls is combined with the use of computer controlled batching plants to achieve precise workability and strength specifications. Even when incorporating small amounts of uncontaminated recycled concrete (typically derived from in-house wastes) the overall quality remains unaffected.

However, in contrast to that, are still present in the construction site workmanship deficiencies which often result in poor execution, rather than inadequacies in the materials themselves.

The consistency of production is evident in the low standard deviations for 28-day compressive cube strengths (f_{ck} = 45–70 N/mm²), which typically remain under 2 N/mm². Table 7 outlines typical mean concrete strengths (f_{cm}), calculated from two or three cubes, at intervals of 20 hours, 3 days, and 28 days. It also includes characteristic cement cube strengths (f_{cu}) based on BS EN 197-1 standards. The data further reveals that even at the 20-hour mark, concrete strengths approach the two-day strength levels for cement, assuming $f_{cu} = f_{cm} - 4$ N/mm².

Concrete specifications follow the guidelines set out in BS EN 206 (BS EN 206:2013). Eurocode 2 (EC2), specifically BS EN 1992-1-1, § 3.1.2 (BS EN 1992-1-1:2004), categorizes concrete strength into predefined classes, often referred to as "grades." These grades indicate the characteristic compressive strength and are expressed either as cylinder strength (f_{ck}) or cube strength ($f_{ck,cube}$ or f_{cu} in BS 8110). The strength class used in design is generally the cylinder strength (f_{ck}), it spans values starting from 20 N/mm² and extends up to 90 N/mm², which correlates to a cube strength of 105 N/mm².

Figure 60a provides a visual representation of the true stress-strain (σ - ε) behaviour for concrete across both normal and high mean strength levels). The constitutive stress‐strain relationship in the design is idealised as a parabola-rectangle in Figure 60b, considering the design strength $f_{cd} = \frac{f_{ck}}{\gamma_m}$, where γ_m is the material partial safety factor of 1.5 at the ultimate limit state.

The strength of concrete $f_{ck}(t)$ within the range of 3 to 28 days can be assessed by testing cubes or cylinders or also by estimating it through a semi-empirical equation. This applies under standard curing conditions and an average temperature of 20°C as follows:

$$f_{cm}(\mathrm{t}) = f_{cm}\, e^{s(1-\frac{\sqrt{28}}{t})} \qquad (13)$$

where $f_{cm}(\mathrm{t}) = f_{ck}(\mathrm{t}) + 8$ N/mm² and s = 0.2, 0.25 and 0.38 for cement class R, N and S.

The strength of concrete at demoulding is determined by the producer based on the manufacturing and handling processes. This is typically measured using the mean compressive cube strength $f_{cm,cube}$. The evaluation generally occurs within 16 - 24 hours after casting. However, there are no established empirical formulas to accurately predict this strength due to its strong dependency on the specific casting and curing conditions.

Cement strength class	Early strength (N/mm²) 20 h concrete	2 days cement	7 days concrete	Standard concrete strength at 28 days (N/mm²)	
	Precaster	BS EN 197	Precaster	BS EN 197	Precaster
42.5N	15–20	*≥10.0*	40–55	*42.5–62.5*	50–70
42.5R	25–35	*≥20.0*	50–70		60–90
52.5N	20–30	*≥20.0*	50–70	*≥52.5*	60–90
52.5R	30–35	*≥30.0*	50–80		70–100

Key: BS EN 197 characteristic value for cement (italics); producer's mean cube strength.

Table 7 Strength requirements for cement based on BS EN 197-1:2011 and typical concrete strengths achieved by precast producers.

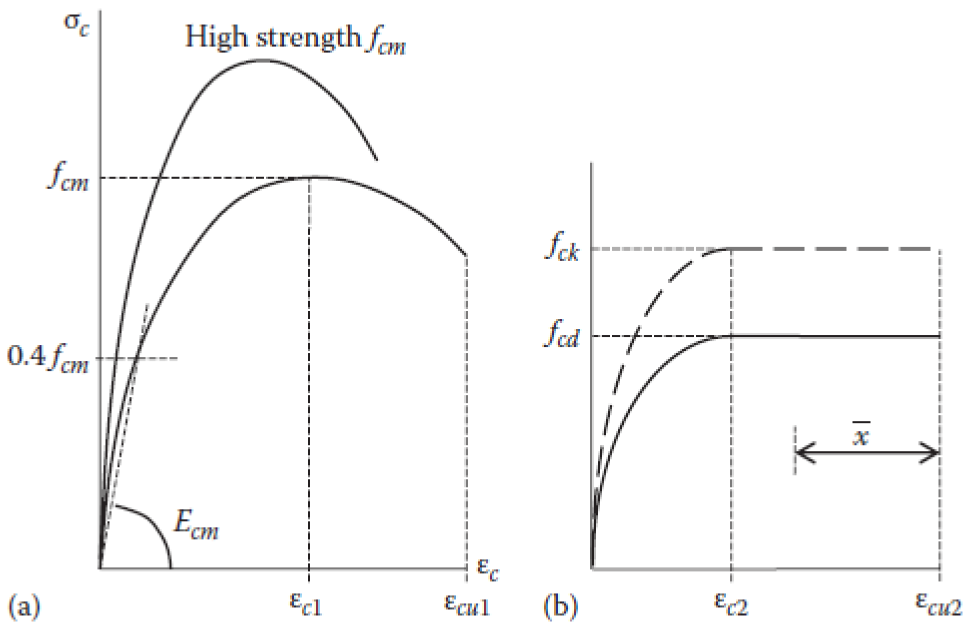

Figure 60 Constitutive stress vs strain relationships for concrete in compression. (a) Actual relationships for normal and high strengths, (b) parabola–rectangle idealization.

In modern precast concrete factories, achieving both optimal workability and rapid early strength development is a crucial requirement. Concrete is generally delivered automatically from a computer-controlled batching plant directly to the moulds, leaving minimal or no opportunity for workability testing. Nevertheless, such tests are typically unnecessary, thanks to the experience and consistency of the batching personnel who tend to maintain mix design consistent and is rarely modified without a valid justification. This kind of approach ensures reliability and simplifies the production processes.

The precast industry gained wide benefits from the developments made in concrete technology in the last century. A key advancement in precast concrete production has been the introduction of self-compacting concrete (SCC). True to its name, SCC flows and compacts under its own weight, removing the need for external vibration during placement (Figure 61). Although SCC initially encountered barriers in achieving sufficient early strength development, these issues were effectively addressed by 2002. Since then, SCC was widely used for creating various precast components, such as structural frames, floor slabs, and staircases.

Figure 61 Self-compacting concrete, flowing and compacting without segregation.

For the production of standard precast elements like columns and beams, concrete is poured into moulds, which are typically made in steel or timber, and occasionally in concrete. These moulds are manufactured to stringent precision standards, ensuring accuracy within ± 3 mm or less in cross-sectional dimensions. Concrete compaction is achieved by using clamped vibrators configured to the optimal frequency based on the size and weight of the mould once filled. This method guarantees proper consolidation of the concrete to get a typical density around 2400 kg/m^3 (excluding reinforcement). The resulting surface finish is smooth and dense, with minimal porosity, significantly enhancing the durability of the final structure.

Concrete grades are carefully selected to maximize the efficiency and structural integrity specific to each type of component. The general provision is:

- C32/40 concrete for flexural components,
- C40/50 for walls in compression,
- From C50/60 to C90/105 for columns.

Surface finishes can vary depending on the particular application and are expressed in [14] as follows:

- “basic” for ground or foundations,
- “ordinary” to receive finishes, but not painted directly,
- “plain” for visual concrete occasionally seen, to be painted directly,
- “special” for architectural finishes.

The Young’s modulus of concrete for gravel aggregate is calculated according to BS EN 1992-1-1, clause 3.1.3, as:

$$E_{cm} = 22\left[\frac{(f_{ck}+8)}{10}\right]^{0.3} \tag{14}$$

For limestone and sandstone aggregates the value should be reduced by 10% and 30% respectively; for basalt aggregates, the value should be increased by 20%.

When the aim is to achieve a composite behaviour by adding cast in situ concrete (of strength f_{cki}) to precast elements, the relative strengths and stiffnesses of the two materials must be considered using the modular ratio method. The most common combinations of in situ-to-precast strengths are f_{cki}/f_{ck} = 20/32 and 25/45. This leads to the modular ratio m = $\frac{E_{cmi}}{E_{cm}}$ ≈ 0.87 as shown in Table 8.

Type of concrete	f_{ck} *(N/mm²)*	f_{ctm} *(N/mm²)*	E_{cm} *(kN/mm²)*	*Modular ratio*
In situ	25	N/A	31.5	N/A
	30	N/A	32.8	N/A
Precast reinforced	30	N/A	32.8	0.96–1.00
	40	N/A	35.2	0.89–0.93
Prestressed	45	3.80	36.3	0.87–0.90
	50	4.07	38.2	0.82–0.86

Table 8 Strengths and short-term elastic modulus for typical concrete used in composite construction.

2.5.2 Mix design

The mix design for precast concrete is determined by the producer in line with the standards established by BS EN 206, which provides guidelines for designing concrete mixes based on performance, composition, and exposure criteria. It involves carefully calibrated proportions of cement, coarse and fine aggregates, water, and various admixtures or additives to address mechanical and durability requirements of the specific precast application (Figure 62).

An important specific consideration in precast concrete manufacturing is early strength development, which facilitates key processes such as early demoulding, prestress detensioning and transportation, often within 12 to 24 hours of casting. To achieve this, the mix typically incorporates rapid-hardening cement types like CEM I 42.5R or 52.5R and is optimized for accelerated curing methods such as steam or warm-water curing.

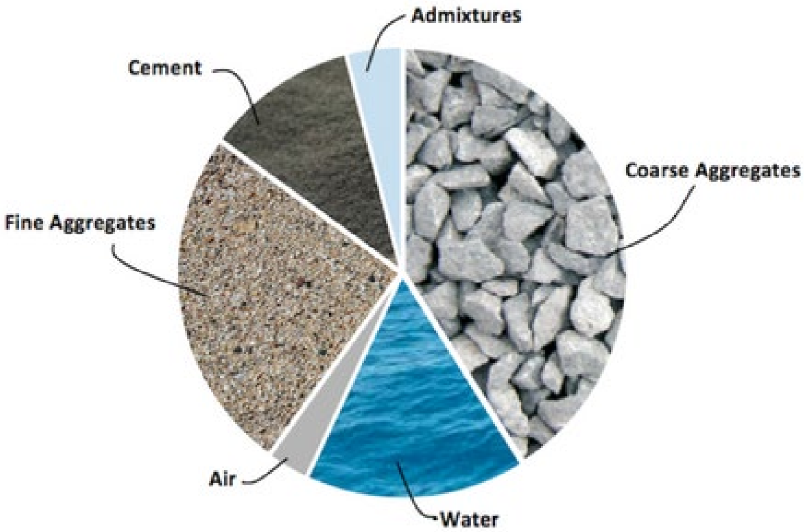

Figure 62 Concrete mix design.

When introducing a new set of concrete mixes, particularly those involving unconventional cement types, novel admixture combinations, recycled materials, or modified curing methods, trial mixes are a mandatory step. These trials are conducted under controlled conditions by the producer and tested to ensure compliance with performance standards, including: fresh mix properties, early strength development within 16 - 24 hours for demoulding or prestressing operations, 28-day compressive strength to assess the conformity to the required characteristic strength and durability parameters (water absorption, porosity, and resistance to freeze-thaw cycles). Moreover, consistency and batch repeatability are measured using standard deviation data collected from multiple samples. The production process will only incorporate a new mix design after it has successfully met all specified criteria during the trial phase.

Cement

Most commonly used cements in precast concrete are Ordinary Portland cement (OPC), rapid-hardening Portland cement (in grey and white), sulphate resisting Portland cement (SRPC), and High Alumina Cement (HAC). These varieties in types are quite distinct either in composition, time of setting, rate of gaining strength and colour characteristics depending on their origin and production.

Despite these differences, all precast cements are required to meet strict national and international standards. These criteria include performance and safety requirements such as strength, set time, soundness, and chemical content. For compliance and traceability, manufacturers must supply Certificates of Test or Compliance for all batches, citing the related purchase order.

The primary requirement in precast work is to ensure a “green strength” to the product, allowing it to be demoulded almost immediately and additionally it must possess sufficient handling and stacking strength within a time frame of 6 to 18 hours.

The second key consideration involves a balance in strength and durability criteria: the material must meet a minimum strength and durability standard, but there must also be an upper limit in order to prevent too much shrinkage, heat generation and costs.

When high early strength is required Rapid-Hardening Portland cement is often employed. This type of cement is more finely ground, which accelerates hydration and enhances strength development at an early stage. Precasters commonly use it during cooler weather conditions and in situations requiring a quick change of moulds and equipment.

Cement designations are complex and many sub-classifications are used [15]. For example, a rapid-hardening Ordinary Portland Cement (RHPC) could be designated by:

CEM I/A-mac.52.5N

where:

- "CEM I" is the main (95-100 per cent clinker) cement type;
- "A" confirms a high clinker content and could be omitted;
- "mac" is an inorganic minor additional constituent at 0-5 per cent;
- "52.5" is the 28-day characteristic cube strength which is specified to be 52.5MPa minimum;
- "N" is a normal rate gain of strength. If the 'N' had been 'R', standing for rapid strength gain, there would be no limits for the characteristic strength.

In some visual concrete applications is used White cement, which offers similar performances to standard Portland cement but with better aesthetical properties. Actually, the raw materials used in white cement are carefully selected to exclude iron traces that produce the grey hue found in other types of cement.

Aggregates

Aggregates are a crucial ingredient of precast concrete, having structural and aesthetic functionalities. They are the main, active element contributing to the weight and basic skeleton of concrete, furthermore providing properties such as compressive strength, deformation, and durability. As aggregates represent the largest proportion of the mix (60 - 80% by volume), their performance affects both the fresh and hardened properties of the concrete.

For precast applications, aggregates have to provide also specific properties in terms of dimensional stability, early strength development and durability under different environmental conditions. Moreover, high quality aggregates can be chosen in the design in order to contribute to achieve particular surface finishes.

The ideal shape of coarse aggregate is rounded, angular and approximately cubic in order to enhance optimum workability of the final mix. Fine aggregates, on the other hand, can range in shape from rounded to angular, depending on the requirements of the precasting process. Some aggregates can be found in nature yet with the desired shape, but mostly of them require further crushing, screening and washing to achieve the optimal characteristics for precast concrete. For example, quarry-sourced materials, which generally start as 150 - 300 mm fragments, often undergo three or four phases of crushing to achieve the desired consistency. The most known natural aggregates are nowadays grouped into flint, volcanic, sandstone, limestone, marble, sands, perlite, vermiculite.

On the other hand, in the market are present also a lot of synthetic aggregates usually reserved for specialized concrete needs such as lightweight, high-density, or architectural applications. These are defined as artificial materials for which the physical properties are tailored for certain uses. They can act as partial or full replacement of natural aggregates and can be produced through a variety of industries processes for satisfying particular performance or sustainability objectives. They can include materials from industrial by-products such as fly ash and slag aggregates, or also polymeric material wastes.

In the last decades crushed concrete has emerged as an economical aggregate in regions where natural aggregates are scarce, providing a sustainable alternative for construction use.

Coarse aggregates are generally stored in their individual size in order to facilitate the selection during mix design. The primary goal of any grading selection is to achieve a structure with minimal voids by filling as much of the void space as possible. This is typically accomplished by combining the largest aggregate size, an intermediate size, and sand or fines. Does not exist general strict rules for gradings, because each specific process has its requirements, so it’s important that the precasters communicate their exact needs to the supplier and ensure that the materials are consistently delivered as specified. Table 9 and Table 10 provide some recommendations.

Size	*Passing sieve*					
	63 mm	*37.5 mm*	*20 mm*	*14 mm*	*10 mm*	*5 mm*
63	85–100	0–30	0–5			
40	100	85–100	0–25	0–5		
20	100	100	85–100		0–25	0–5
10	100	100	100	100	85–100	0–25

Table 10 Nominal single size coarse aggregates.

Type	*Passing sieve*					
	5 mm	*2.36 mm*	*1.18 mm*	*600 μm*	*300 μm*	*150 μm*
1	90–100	60–95	30–70	15–34	5–20	0–10
2	90–100	75–100	55–90	35–59	8–30	0–10
3	90–100	85–100	75–100	60–79	12–40	0–10
4	95–100	95–100	90–100	80–100	15–50	0–15

Table 9 Suitable fine aggregate gradings.

Aggregates can be subdivided in three main categories depending on their specific gravity (SG):

1. Normal weight aggregates (NWA): are the most used in the precast industry due to low cost and good mechanical properties. The term “normal” means simply that no specific type of aggregate is defined, as opposed to light and heavy aggregate. They have a specific gravity of between 2.0 and 2.6 (2 to 2.6 times heavier than water). This results in an apparent density of around 2200 - 2600 kg/m³, which is well-suited for making concrete having a normal unit weight of approximately 2400 kg/m³.
2. Low density aggregate (LWA): are produced by the use of lighter materials such as expanded clay, expanded shale or expanded slate in concrete to decrease the self-weight of the concrete. They usually have SG ranging from 1.2 to 2.0. The use of LWA brings to the production of lightweight concrete, with unit weight of around 1400 - 2000 kg/m³.
3. High density aggregate (HWA): are specialised materials used for radiation shielding, counterweights and security applications. Their specific gravity is normally within the range of about 3.5 to 6.0, much higher than that of conventional aggregates. Their application in the precast industry is not particularly relevant, but it may be expected to increase due to improvement in the nuclear industry for the generation of electricity.

Water

Water is one of the cheapest elements in the whole mix design process, but its concentration cannot be ignored because is responsible of macro effects in the fresh and hardened concrete. Water is an essential element for precast concrete manufacturing in many aspects, as it is involved in hydration activity and also significantly impacts workability, strength development and durability of the product. Indeed, the primary purpose of water is to start the chemical reaction of cement with particles and then form the gel (called calcium silicate hydrate, C-S-H) that is responsible for the mechanical strength and for binding the concrete. Otherwise, the water-cement ratio must be low in prestressed concrete to provide increased early strength and increased durability.

Concrete strength and durability are largely influenced by the porosity of the hardened concrete; furthermore, the porosity of the paste is primarily determined by the extent to which the voids between the cement grains have been occupied by the products of hydration. This of course is related to the volume of hydration products and to the volume of pore space to be filled [16]. Figure 63 shows that increasing water content relative to cement one will increase also the average spacing between cement particles. The challenge of the water-cement ratio lies in its dual impact: while a higher ratio provides more water to sustain hydration, it also increases inter-particle pore space. This greater pore space subsequently demands a higher level of hydration to achieve effective pore-filling. High-performance concrete overcomes this by reducing the water-cement ratio to approximately 0.35 or even lower in some cases. This approach minimizes the water volume between cementitious particles, although complete hydration, where all cement converts into hydration products, cannot be achieved. Despite this limitation, if the inter-particle pores can be sufficiently or completely filled with hydration products, the goal of a low-porosity concrete can still be attained, even though some unreacted cement will inevitably remain.

It’s important to heavily control the water content in the precast industry, and this is often done with admixtures like superplasticizers that increase workability without adding excessive water, because the ability to achieve rapid strength and consistent quality is key to efficient production cycles.

In addition, appropriate curing methods, which usually include the provision of a suitable level of moisture and temperature inside curing chambers, are employed to encourage hydration to proceed to completion and to minimise cracking caused by shrinkage.

Water also influences the thermal response of concrete, with too high evaporation leading to early-age cracking, especially in thin precast items.

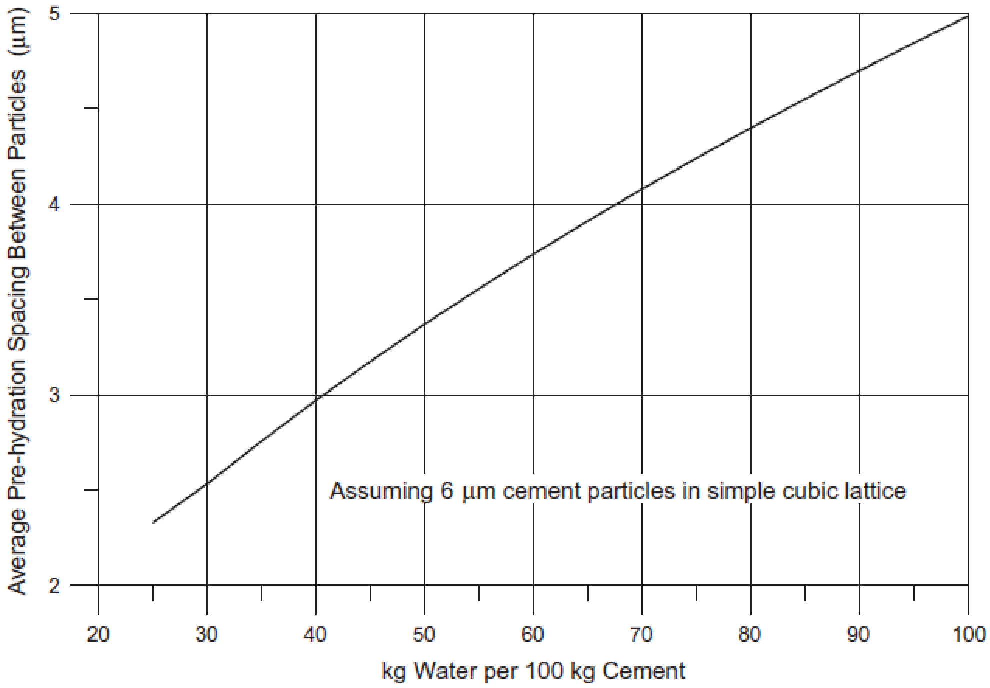

Figure 63 Relationship between water content relative to cement content and average spacing between cement particles.

Admixtures

Admixtures are mixed with concrete in order to improve its performance, during production and handling and during the final shape. So, they are essential to optimize the processes of production in controlled environment and also to ensure good performances in service.

The old adage that a well-designed concrete mix doesn’t need an admixture was very common, but today can be assessed that is not true. There are, in fact, some properties that can only be achieved by the use of admixtures, both in fresh and hardened concrete.

Some precasters, still today, avoid to incorporate admixtures due to the additional complexities in mix design and quality control they introduce, which can result in unpredictable outcomes. Such attitude comes also probably from the past issues of corrosion and lack of durability due to chloride-based accelerators. Since 1977, chloride-based admixtures have been prohibited in reinforced and prestressed concrete applications. Alternative chloride-free accelerators are now available; while they present higher costs and varied performance characteristics, they offer safer options [17].

When used appropriately under controlled conditions, modern admixtures can enhance both fresh and hardened concrete properties, providing big improvements in the production processes and in the in-situ work. Trial mixes are crucial for determining the most suitable option for specific production needs.

Focusing on the precast industry, there are some main groups used:

1. Retarders: are applied to precast production in two main ways. Firstly, they can be used as an element to delay the stiffening and hardening processes to reduce the heat of hydration developed. Secondly, they are applied in the surface of the moulds, in order to delay the hardening of concrete, being able to remove a layer of material and expose the aggregates to get a final aesthetical effect for architectural applications.
2. Workability: Chemicals based upon lignosulfonate, carboxylic acid, naphthosulfonate, formaldehyde and similar are used in concentrations ranging from 100 ml to 1000 ml per 50 kg of cement. When dealing with high-range water-reducing (HRWR) or self-compacting concrete (SCC), this range is typically doubled. These admixtures should be added simultaneously with the mixing water. It is strongly recommended not to introduce these additives into dry mix ingredients, as this practice can lead to coating of the cement particles, which may inhibit hydration and result in unwanted retardation.
   These substances work by creating a strong dipole moment, negative charges are placed on the surface of particles, resulting in their mutual repulsion. The three primary functions of workability aids include enhancing workability at the controlled water-to-cement ratio, preserving the controlled workability at a lower w/c, or a combination of both. Additionally, there is a fourth benefit, which is an improved surface finish characterized by smaller surface air voids, a feature that is advantageous for aesthetically pleasing concrete finishes.
   At higher concentrations these products are known as superplasticisers and self-compacting mixtures, in order to improve workability without increasing water. As said before these are used mainly for high range water reducing and self-compacting concretes.
3. Waterproofers and water-repellent: are products designed to prevent the passage of water through the matrix, ensuring their effectiveness even under high hydrostatic pressure. Since concrete has a porous matrix and the solid aggregates can have a certain permeability, these admixtures have to act in a double way. They have to block the pore structure and apply a highly effective water repellent layer to all surrounding solid material particles. This dual objective is generally achieved by combining a swelling, water-activated bituminous compound with a stearate-based powder that reacts with free lime to produce the water repellent metallic calcium stearate. The primary benefit of making concrete water repellent is that visual concrete maintains its appearance intact for many years. A challenge associated with water repellent products is the jointing of mortar, as the surfaces tend to repel the mortar. This issue can be resolved by adjusting the mortar mix with approximately 1 percent cellulose solution instead of using plain water.
4. Pigments: are described as a fine dry powder, a slurry, or an aqueous suspension of that powder, which is inert to all other components and is designed to impart a specific colour to concrete. They are available in various particle shapes and sizes, however they all share the

common characteristic of having a particle size 10 to 100 times finer than that of cement. The colouring mechanism involves the distribution of these particles over the cement and fine aggregate fractions, with some influence on the coarser aggregate particles. It is evident that the colour of the cement and fine aggregate significantly affects the final product's colour, necessitating careful selection.

Air-entraining admixtures have not been mentioned in this list because, despite their huge use in wet-cast concrete, in precast production they are not so common. This is because freezing thaw cycles resistance is generally gained during the production process by using a higher cement content than in situ concrete. In frost resistance achieved through the use of these additives is function of the bubble size distribution, dimension and spacing, so in mix with a high cement content (above 350 kg/m^3) becomes difficult to allow the entrance of the correct dosage of air.

2.5.3 Reinforcement

Reinforcement is generally used for three main purposes in precast concrete:

1. Structural: is the main flexural reinforcement used to carry vertical and horizontal loads. It will compensate the weakness of concrete in tension and increase the overall performance of the structure. Moreover, it is used also to limit the width and propagation of cracks in traction zones of concrete.
2. Handling: is the reinforcement added specifically to resist stresses during lifting, transport and installation activities. It's additional respect to the flexural one and is not used to improve the structure's long-term performance. The stresses arising from these processes must be carefully evaluated, because are in general significantly different from those during service life. They depend mainly on the lifting method, orientation during lifting operations, position of the lifting supports, load combinations and dynamic effects induced during the transport stages.
3. Shrinkage: is the reinforcement mainly applied to minimise and control cracking due to volume variation in concrete. It is especially important in elements that have one face exposed to natural weathering and the other subjected to continuous dump conditions. It can be the case of some wall panels, slabs, pipes, beams and precast facades.

Reinforcing bars and meshes

Precast concrete members can be heavily reinforced if required, since they are cast horizontally and no lifting is involved. The code permits up to 4% of the cross-section generally and 8% at laps, but this is seldom applied in favour of higher concrete strengths. The most used rebars are the hot rolled high tensile ribbed bar (HT bar) designated "H". For the design is used the idealised bi-linear stress-strain relationship with inclined or horizontal branches, as shown in Figure 64. In general, the horizontal branch is more convenient because has no limitations of strain in under-reinforced sections. Mild steel is often used for projecting loops etc. because it is easier to hand-bend on site.

Bar diameters commonly used are 8 and 10 mm for column links, 10 and 12 mm for beam stirrups and other distribution or anti-crack bars, and 16, 20, 25, 32 and 40 mm for main flexural bars.

One of the most important properties is the bond strength. New steel generally is made to meet bond specifications, but the bond can be further improved by allowing the steel to develop a slight rust before utilization, it will enhance the mechanical interlock. It is essential to avoid using steel that has significant rust on its surface, and such steel should be subjected to sandblasting to eliminate the contaminants.

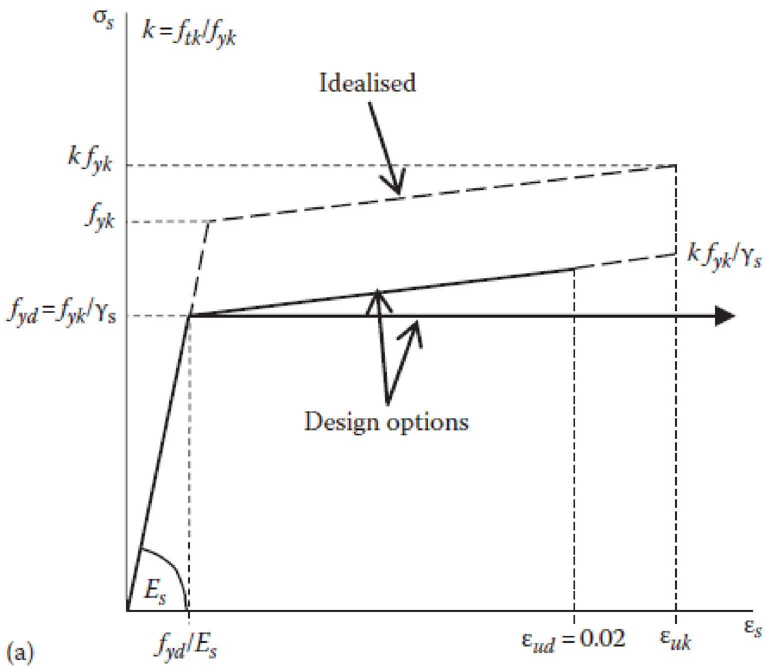

Figure 64 Idealized and design stress vs strain relationships for reinforcing bars.

The protection of steel is essential for storage in humid, marine, or chloride environments, as well as when utilized as reinforcement in exposed environments. This can be accomplished through methods such as galvanising, applying a styrene-butadiene cement slurry, or bitumen dipping, among others. Steel starter bars or any steel that extends from a precast unit must be coated with a cold zinc coat, cement slurry, or a styrene-butadiene cement slurry. Such measures ensure that the steel remains in an almost pristine state and prevent rust from washing down onto the concrete.

Steel meshes are used generally as reinforcement in flat units such as slabs and walls. The most used mesh size is or 7 mm bars at 200 mm centres in both directions, but it deeply depends on the loading conditions and service use of the structure. The Young's modulus is taken as 200 kN/mm^2.

Fibre reinforcement

Fibres and meshes composed of steel, polypropylene, glass, and carbon are currently utilized in various types of precast products. Their primary applications include concretes that necessitate thin sections, impact resistance, or specific thixotropic properties. It's common to consider only carbon and steel fibres as effectively reinforcing elements due to their strengths and moduli exceeding those of concrete, while polypropylene and glass possess moduli that are either lower or comparable to that of concrete.

Steel fibres find application in particular products that demand a combination of high strength and impact resistance, such as explosion-resistant units. However, these fibres necessitate careful handling with steel-faced gloves and should be added slowly to a working pan mixer; otherwise, they may clump together or form spheres. To achieve adequate workability, additional water and/or the incorporation of workability admixtures is essential, and the compaction through vibration must be more energetic than that required for a conventional mix.

Polypropylene and the other plastics fibres being developed give good impact resistance and crack control to precast products.

Currently, carbon fibre is quite costly; however, it stands out as the most appealing of all fibres due to its remarkably high strength, as well as its excellent bonding and corrosion-resistant characteristics. The cost may potentially decrease with advancements in manufacturing techniques, yet even at its current price point, it could still be suitable for certain products.

Reinforcement spacers

Spacers are intended to ensure the required minimum cover between the steel reinforcement and the mould, and they can be constructed from various materials, including:

(a) Concrete or mortar blocks;
(b) Plastics, typically filled polyethylene or polyamide;
(c) Steel featuring end-leg protection.

There are two primary types of spacers [15]:

- T1 Trestle: Primarily supports horizontal bars (Figure 65)
- T2 Ring: Supports horizontal, vertical, or angled bars (Figure 66)

Figure 65 Trestle spacer made in concrete to support horizontal bars.

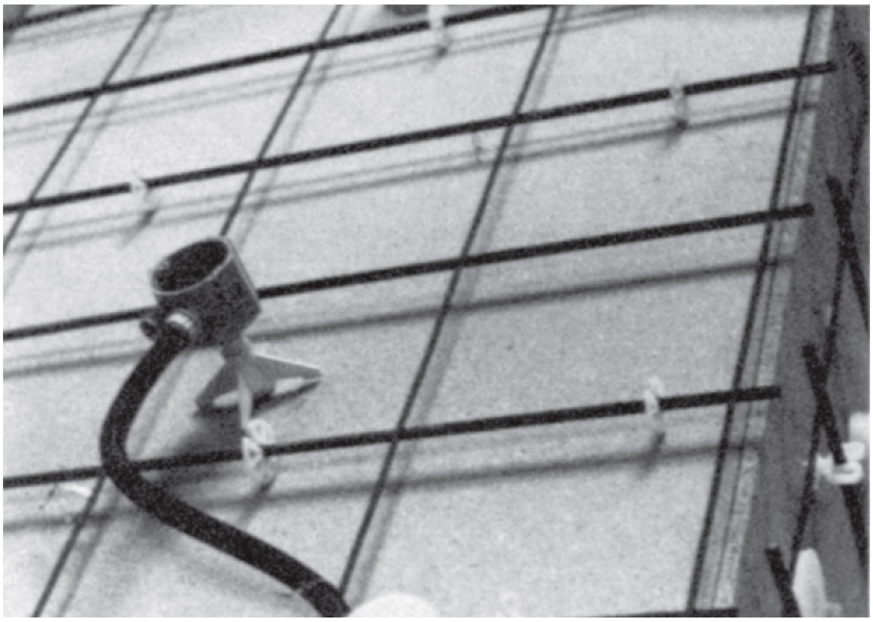

Figure 66 Ring spacer made in plastic material.

2.5.4 Cast in fixings and fittings

One of the principal advantages of the use of precast elements in the construction site is the simplicity of installing services within the finished structure. To ensure this feature the detailing, scheduling, procurement, and installation of various items represent a crucial component of the production process. In general, the choice and the position of the fixing systems is discussed between the manufacturer and the engineer. The range of fittings and fixings available includes [17]:

a) channel fittings for masonry and curtain walling and embedment for masonry fixing;
b) slots for brick ties;
c) column corner protection, nosing to stair treads;
d) fixing blocks and battens for windows, door lining fixing and so on;
e) hangers and anchorages for services and false ceilings;
f) bar couplers for continuity steel, also reinforcement connection systems;
g) projecting bolts and studs;
h) sleeve anchors, pins, ties and bats in sandwich elements and for subsequent dry lining and insulation;
i) tying wire, black annealed and stainless steel;
j) sockets or bolts for attachment of props and bracings for use in site erection;
k) a range of fixings for embedment or subsequent installation for attaching connections, services and so on.

The load bearing capacity of these elements depends a lot on the strength of concrete on which are embedded and on the degree of precision during the installation. The main evaluation of the capacity is done when concrete reach 28-days strength, because at earlier stages the capacity is drastically reduced.

Fixings are required to be cast-in within specified tolerances, ensuring that there is no displacement during the compaction process or damage during the mould stripping and subsequent handling processes. It is essential to take precautions to avoid closely set lines of inserts, as these can create stress inducers and lead to cracking. The distances from the edges and the spacing of fittings must be arranged so that the area of the resisting concrete shear cone is sufficient to accommodate the service loading on the fixing, along with an appropriate safety factor. The locations of the fixings should be planned with consideration for the positioning of reinforcing bars within the element. In-shop welding to embedded fixings is prohibited, as this may cause embrittlement of the materials used in the production of certain types of fixings.

Many erectors prefer to use post-fixed fitting for propping purposes; they fall into several categories:

a) expanding fixings where shells, collars or rubber sleeves are forced against the wall of a drilled hole by tightening the attachment nut or bolt;
b) self-drilling and toothless fixings, expanded by plug when the fixing is driven into a hole;
c) fixings where the shell is expanded by driving a plug using a setting tool through the threaded part of the fixing;

d) ‘through fixing’ where a fixing can be installed through the fixture, the fixture thus providing a template for the drilling operation.

2.6 REGULATIONS AND REFERENCE STANDARDS

Precast concrete structures are governed by a complex range of regulations that vary depending on the region and type of structure. In this section will be provided a general and synthetic overview of the main reference standards used in the different stages of design, production and construction.

The study given in this thesis is directed to an international read and so is mainly referred to EN 1992 and BS EN 1992. In various sections is still used BS 8110 due to references in some projects.

In the last chapters will be examined a case study of a building located in Italy and so the whole section will take as reference the national code NTC 2018 (Italian National Annex of EC 2).

Design phase

This phase involves structural analysis, material selection and documentation of specifications. Design standards play a crucial role in aligning the structural design objectives with the required physical properties to be realized during production. The resulting outputs are in general drawings and specifications and are directly used in the manufacturing process.

1. ACI 318 (USA) - Building Code Requirements for Structural Concrete: it is the primary structural design standard for reinforced and prestressed concrete in the United States and establishes comprehensive requirements covering strength, serviceability, durability, and detailing. It is widely adopted into local building codes such as the International Building Code (IBC), moreover it is applicable to both cast-in-place and precast concrete systems.
2. Eurocode 2 - EN 1992: it regulates the structural design of buildings and civil engineering works made from concrete. It addresses both ultimate and serviceability limit states for the design of precast elements and their connections. National annexes enable the code to be customized to local needs.
3. Eurocode 2 - BS EN 1992: the UK adopts Eurocode 2 with its National Annex, modifying safety factors and material parameters for UK-specific conditions.
4. ISO 16310 - Performance Requirements for Structural Precast Elements: it outlines structural, serviceability, and durability performance requirements for precast elements, intended for international adaptation.
5. NTC 2018: Italy's national technical regulations for structural design by considering Italian safety factors and seismic criteria.

Production phase

This phase transforms the design documents and projects into physical entities with observable quality ensuring that units produced meet the desired design performance. It is a high-quality process performed in factories and so subjected to strict levels of control.

1. PCI MNL 116/117: these manuals are used is the United States to provide quality control specifications for structural (MNL 116) and architectural (MNL 117) precast concrete

production. They include requirements about inspection processes, materials, plant certification and dimensional tolerances.

2. ASTM standards: are used in the United States to ensure material quality and performance. Some examples are ASTM C150 (cement), C33 (aggregates), and C494 (admixtures).
3. EN 13369 - Common Rules for Precast Concrete Products: provides general information, performance and rules for all the precast products. It represents the base for these standards and so must be accompanied by specific codes for each precast element.
4. ISO 22965: defines the requirements for production at an international level. It is subdivided in sections to cover all the aspects related to precast concrete, in particular during the production phase the most important are part 1 and 2.
5. UNI Standards: are the Italian version of the EN standards commonly used in Europe.

Construction phase

This stage concerns transportation, erection, final integration and assemblage of components into the structure. Standards in the construction phase ensure that the precast elements are set up safely, from the lifting to the tolerances. This phase is closing the control loop to the design specifications and guarantees that the system as a whole meets the desired performance.

1. ACI 301 (USA): is complementary to ACI 318 (United States) and involves the specifications for site construction such as mixing, curing etc.
2. EN 13670: checks the correct execution practices in the construction sites for the erection of concrete structures.
3. ISO 22965: is the main international reference in the construction phase, especially part 3 to check post installation conformity and performance validation.
4. NTC 2018: applies in Italy to regulate site inspections, load testing, anchorage, and seismic performance verification.

3. PRECAST BEAMS

3.1 INTRODUCTION AND TYPOLOGIES OF PRECAST BEAMS

Beams are the main horizontal load-carrying members in skeletal structures. They are, by definition, relatively small prismatic sections of large flexural (typically, 300-800 kNm) and shear (100-500 kN) capacity [4].

Precast beams can fall in different categories. A first deep classification can be made between fully precast beams (PC) and composite steel trussed concrete beams (CSTCB). This distinction lies basically in the material difference and in the construction methodology.

Fully precast beams are structural elements made entirely within a controlled factory by using reinforced or prestressed concrete (Figure 67). These elements are computed and designed in order to be lifted and placed on site without the need of additional in situ concrete casting, making them well suited for fast construction projects and consistent quality control. Their solid section of concrete delivers good performance in compression and bending, but their mass can pose limits for transport and erection, especially for longer spans.

Figure 67 Fully precast concrete beam laying on two precast columns.

On the other hand, composite steel trussed concrete beams are formed by a hybrid system (Figure 68): a prefabricated open-web steel truss, typically in a triangular or V-shaped configuration, combined with cast-in-place concrete. In this way it is able to achieves the beneficial properties of steel in tension with high strut capacity of concrete in compression producing a very effective structural element for spanning longer distances with less self-weight overall as compared to fully concrete beams. The main drawback of this system is that it needs a portion of concrete to be casted in situ, and so it requires a more complex onsite coordination that will inevitably lead to higher construction time. Despite that, this system is today used a lot in particular in long-span applications including industrial roofs, bridges and large open halls where enhanced structural properties are fundamental and minimizing dead load is necessary.

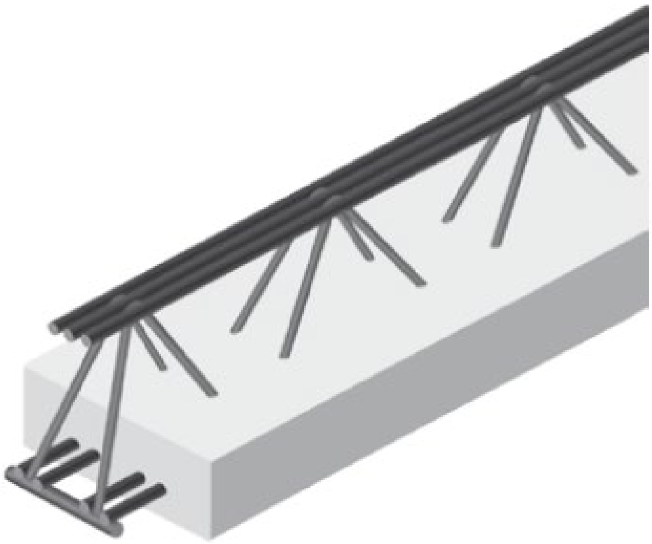

Figure 68 Composite steel trussed concrete beams (CSTCB).

The present thesis will focus mainly in the detailed analysis of CSTCB. Nevertheless, for the sake of completeness, in this introductory section a brief description of the fully precast beams is given.

Fully precast beams

Fully precast beams fall into two distinct categories: internal and external [4]. Internal beams typically have symmetrical loading, with floor slabs positioned on both sides of the beam, resulting in a symmetrical cross-section as depicted in Figure 69a. The design often prioritizes minimizing depth to maximize headroom, so to enhance structural efficiency, internal beams are frequently pretensioned. In some cases, a portion of the beam may be recessed into the depth of the floor slab to create what is known as an inverted-tee beam. Conversely, external beams, are asymmetrically loaded and generally have L-shaped cross sections. When the floor slab rest on the bearing nib, torsion occurs and so must be carefully accounted for in the design process. Tall edge beams are known as spandrel beams and are reinforced against overturning by projecting rebars anchored to columns (Figure 69b). Spandrels commonly serve the additional purpose of creating a dry envelope around the building's perimeter by functioning as temporary weather shields between consecutive storey levels (Figure 70). Unlike other beams, edge beams are not pretensioned due to their asymmetrical shape, which eliminates the need. Furthermore, since beam depth does not impose limitations, pretensioning is generally unnecessary.

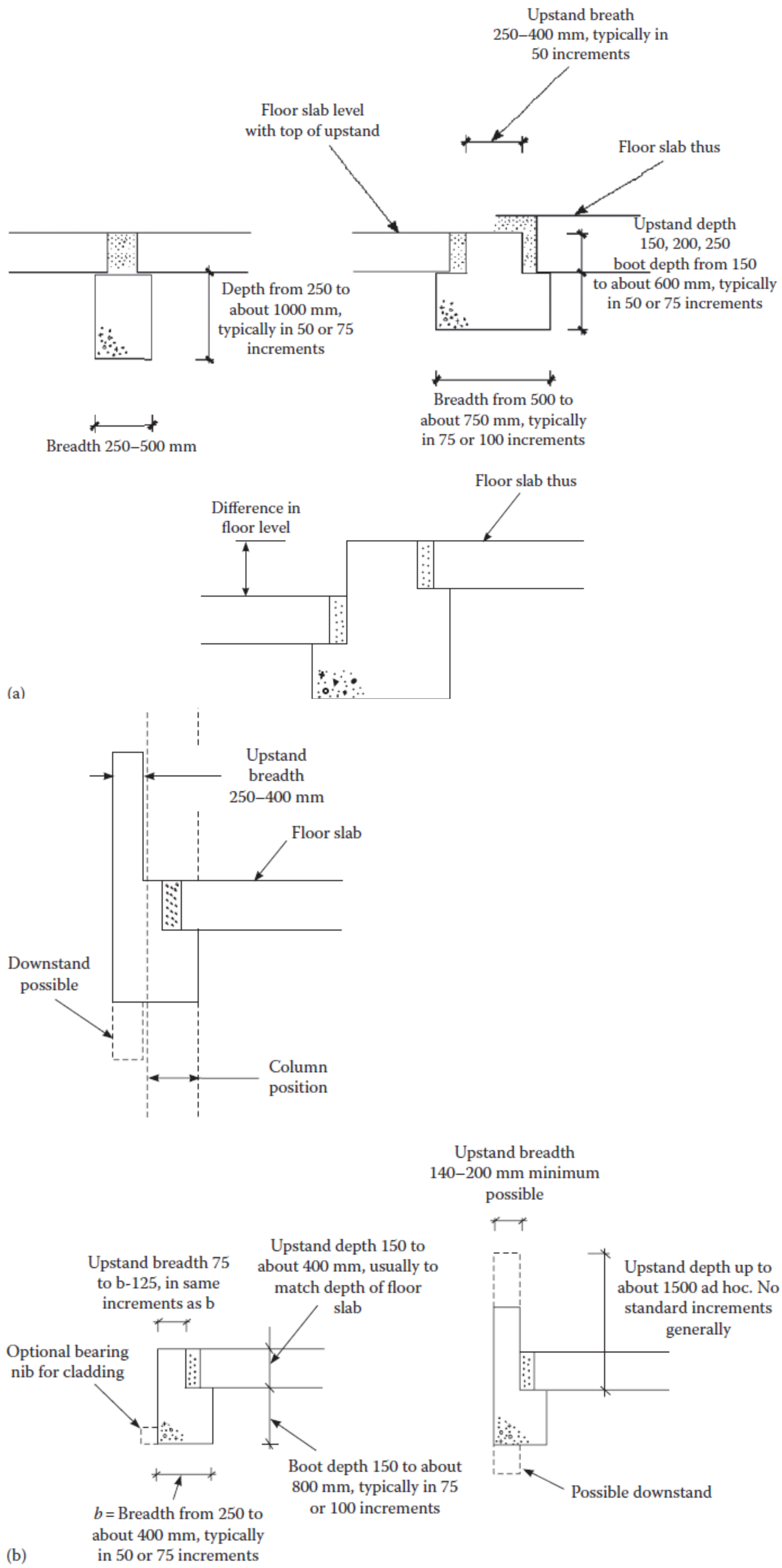

Figure 69 (a) Internal rectangular and inverted tee beam. (b) Edge L and spandrel beam.

Figure 70 Edge spandrel (deep L-beams) supporting hollow core floor units. Site construction dowelled onto a column head with corbel.

L-shaped edge beams are designed to support asymmetrical floor loads. The portion of the beam that provides direct support to the floor is referred to as the "boot," while the vertical section is known as the "upstand." Two main types of edge beams are present:

1. Type I: Features a wide upstand integrated as part of the structural section (shown in Figure 71b).
2. Type II: Includes a narrower upstand that serves as permanent formwork for the floor slab and is treated as monolithic with the in-situ concrete infill at the slab's ends (depicted in Figure 71a).

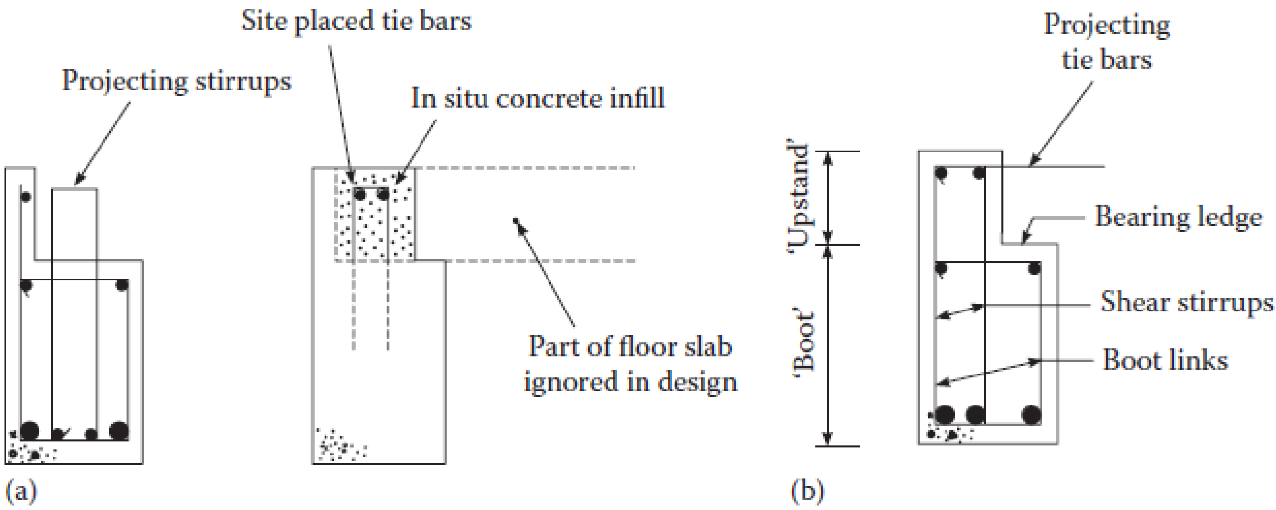

Figure 71 L-section edge beams: (a) composite and (b) non-composite.

The design of these beams adheres to standard principles of reinforced or prestressed concrete, accommodating specified load conditions and support configurations, which may be either simple or continuous. Unlike monolithic reinforced concrete design, where cross sections and reinforcement are specifically engineered for a project's requirements, precast concrete relies on a reverse approach. Manufacturers select standard predesigned beam sections tailored to suit the majority of building requirements. For each beam size, optimal flexural and shear reinforcements are calculated to minimize material use. These standardized designs focus primarily on variations in depth, width, and reinforcement volume. Simple computer software or spreadsheets are typically employed to streamline these calculations, ensuring efficient production of precast beam designs.

3.2 COMPOSITE STEEL TRUSSED CONCRETE BEAMS (CSTCBs)

3.2.1 Historical Notes

Mixed structures were first introduced in the early 1900s, achieved by integrating metal profiles into concrete. This approach facilitated effective collaboration between the two materials through extensive bonding surfaces. During the 1930s, the construction system underwent significant evolution, reducing the role of concrete to a slab connected to metal beams using anchors. Its adoption gained push after being recognized by the AASHO in 1944 and subsequently included in German DIN standards. Over time, interest in this technology grew, driven by theoretical advancements and experimental studies. These developments allowed for material optimization, the creation of innovative construction techniques, and the simplification of processes that impacted production costs. As a result, this construction method ultimately became a competitive alternative to traditional techniques, particularly in the field of bridge construction.

At this point, in 1962, Engineer Leone thought of transferring the mixed structure technology from infrastructure to residential construction, optimising it in relation to the required performance [18]. He patented the first composite steel trussed-concrete beam (CSTCB) calling it "Trave REP®" (most diffused commercial name of CSTCBs). Its idea was to remove the core and the upper plate of traditional beams because they were exuberant from the performance point of view and this led to the minimization of the production time.

The first CSTCB beam was idealised with a single web and a large plate (Figure 72). This was improved through the doubling of the webs and the anti-slip support device, a substantial overcoming of the previous limits, allowing them to be used in more complex constructions such as bridge beams and beams with large spans (Figure 73).

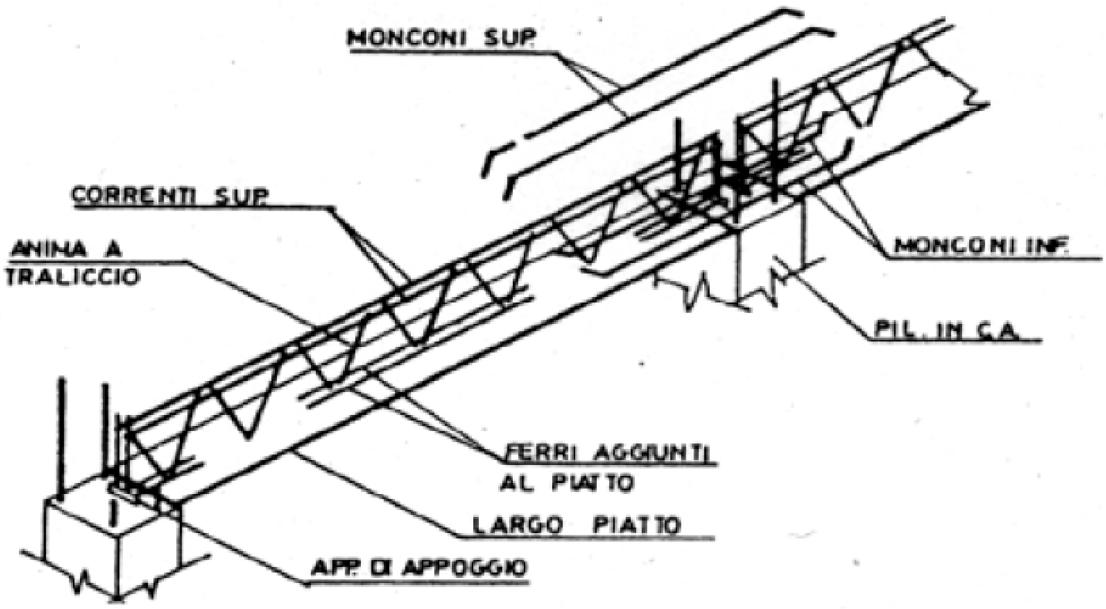

Figure 72 Self-supporting beam REP® patent.

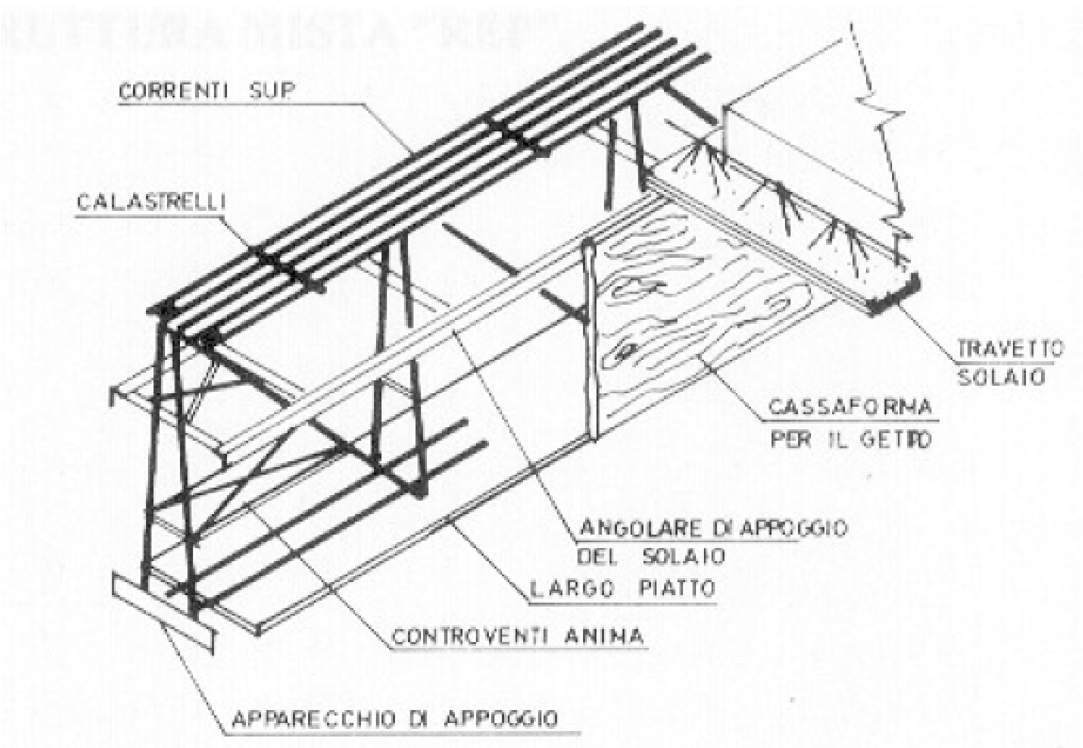

Figure 73 Improvements on the first self-supporting beam REP® patent.

In the following years this type of technology started to became more popular in Italy and also in the rest of the World, mainly due to easy of production and high performances achieved.

A similar construction technique called "Slim Floor" appeared in Scandinavia in the Eighties [19]. Likewise the CSTC beams, Slim Floor girders feature a steel beam embedded in cast-in-place concrete with a lower wide flange plate used to support the slab during initial construction phases. It differs from CSTC beams in their design, employing solid steel sections, either wide flange or rectangular hollow sections, and specific shear connectors. These connectors often take the form of shear studs welded to the top flange or raised patterns rolled onto the beam's surface, enabling composite action.

Another type of composite girder, known as the FRC-encased steel joist composite beam, was introduced in the United States during the late 1990s for seismic-resistant frames [20]. This innovation features a plane steel joist made from angle and plate profiles that are encased in concrete reinforced with fibres. This fibre reinforcement eliminates the need for stirrups while maintaining section integrity. Notably, the plane joist does not need to support the slab before the concrete is cast in place. Analytical studies examined flexural and shear strength without utilizing web reinforcements. However, the influence of chord angles on shear strength, both independently and in conjunction with the concrete section, was not experimentally validated.

In recent years, CSTC beams have drawn significant attention within Italy, leading to numerous theoretical, numerical, and experimental investigations. This growing interest stems from several factors: the need to enhance the understanding of their resistance mechanisms, compliance with safety and serviceability requirements outlined in updated construction codes, efforts to expand their range of applications, and advancements in creating custom-designed trusses.

3.2.2 General Description

Composite steel trussed concrete beams (CSTCBs), also called hybrid truss beams, have widely increased their popularity in the last years due to their ability to offer rapid construction processes, reduced on-site labour demands, and outstanding economic advantages.

These hybrid beams are fully prefabricated mixed beams consisting of a steel lattice girder, composed with concrete. The individual components of the truss are assembled using a welding procedure under CO_2 protection gas.

CSTCBs have not to be confused with common composite structures. In both cases the collaboration between steel and concrete is the crucial feature, but this engagement works with in completely different static way in reinforced concrete structures and in hybrid systems. E. Giangreco [21] described the behaviour of a RC structure as a modern family mènage “where the two spouses collaborate to the extent that they are able to take on the external burdens, their effort increases as the work overload increases and the joint effort goes on until their resources are exhausted at the same time or at a later time until one reaches the collapse conditions already reached by the other” (Figure 74). Instead, in mixed structures the two “partners” use each other's resources at different times in order to meet the needs of the resistant element they have formed together. Initially, the concrete lives off its income, the metal structure bears the external burdens alone, until it reaches maturity and has the capacity to take on part of the external weights and part of the increased workload. The most characteristic difference between reinforced concrete beams and composite structures is already apparent. The reinforcements of reinforced concrete structures do not have independent load-bearing capacity and are unable to perform any function before the concrete casting. On the other hand, CSTCBs are already finished elements which have an initial load-bearing capacity independent from the association of the two components (Figure 75). Only in the operational phase steel and concrete cooperate in the strength of the whole as a single element.

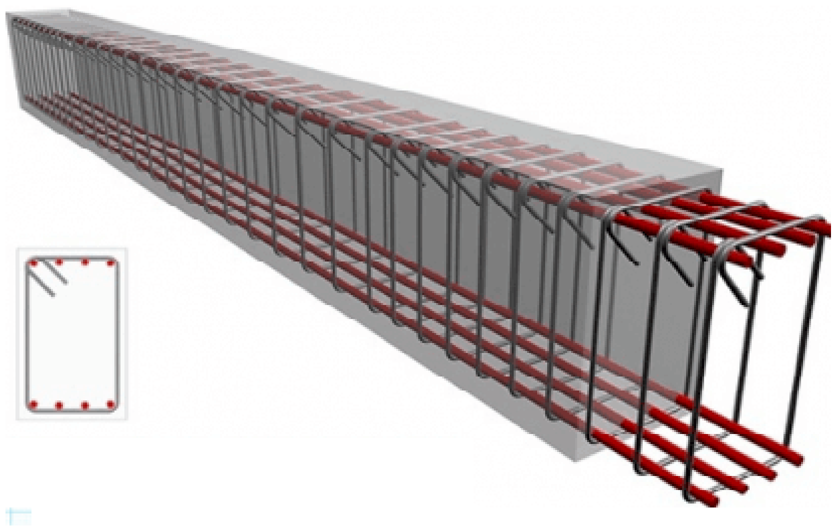

Figure 74 RC beam element.

Figure 75 The self-supported prefabricated steel truss.

The primary steel truss used in these hybrid beams is constructed by welding together steel components, plates, straight bars, and curved diagonal bars. A common configuration involves using a steel plate as the bottom chord, straight bars as the top chord, and bent bars forming diagonal truss members. The design creates a characteristic triangular cross-section, enhancing stiffness against torsion and out-of-plane buckling while ensuring robust load-bearing capacity for supporting slabs. Later advancements modified this design by replacing the steel plate bottom chord with precast reinforced concrete bases. Here, diagonal bars are welded onto straight reinforcement bars embedded in the precast concrete base, introducing added structural innovation. (Figure 76)

Figure 76 CSTC beams laying into a steel column.

From the previously given description can be said that CSTCB stand between reinforce concrete and composite steel concrete typologies. Since the truss have to withstand alone the loads before the casting of in situ concrete, it’s possible to distinguish between two phases of the life of the structure:

- Phase I, before the hardening of the concrete. Only the steel members are responsible for the mechanical resistance.
- Phase II, after the hardening of the concrete. The concrete core and steel truss contribute together to the mechanical response of the composite beam.

The principal characterising element of the CSTCB before the casting of concrete is the steel truss. It is made principally by the lower chord, the bottom cord and the core or web [22]. The bottom chord may consist of a steel plate, possibly accompanied by longitudinal bars, a bottom plate of reinforced concrete or steel bars only (Figure 77). The upper current may consist of round, square or angle bars, welded to the wall bars constituting the web. The web is made up of a simple or multiple lattices of round bars, generally of equal diameter.

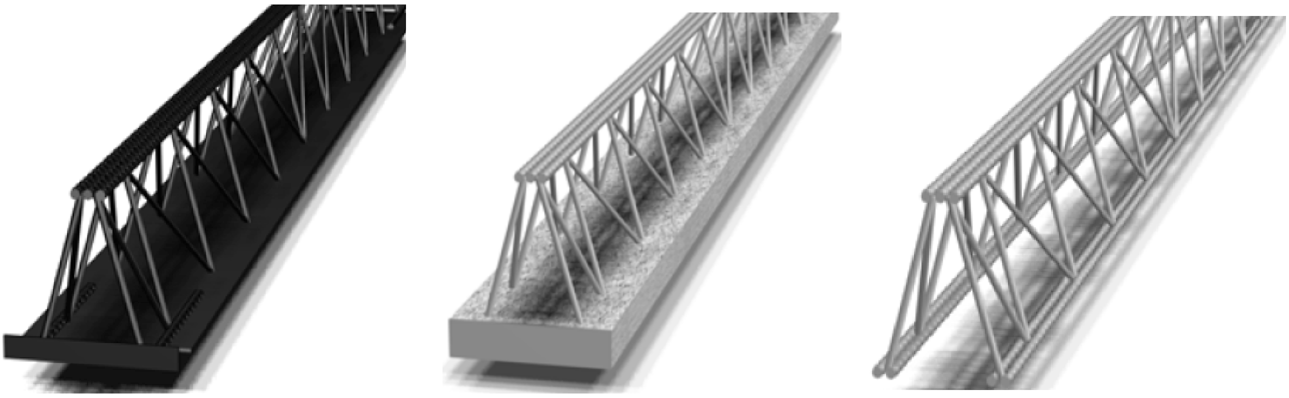

Figure 77 (left) Beam with bottom steel plate; (centre) beam with concrete bottom chord; (right) Beam with bottom chord made only by steel bars.

In order to achieve structural continuity in the second phase are used supplementary upper and lower bars, metal trusses positioned across the nodes and eventually sleeves. For constructions in seismic zones, stirrups are used in the critical zones as required by the technical standards.

The main advantages given by the use of this system can be summarised as follows:

- Reduction of installation times and consequent reduction of overhead costs;
- Elimination of the use of formworks and props due to self-bearing characteristic of the beams, resulting in greater site cleanliness and organization (Figure 78);
- Reduction on dimension of cross sections and reduction of weights discharging into the foundation;
- Reduction of long-term deformations due to high stiffness of beams;
- Reduction of risks associated with site work on beams and no need for skilled labour;
- Improved versatility of the project, beams can be modelled according to specific structural requirements;
- Increased spacing between columns with a consequent increase in the span of beams.

Figure 78 Picture from construction site that highlights the absence of formworks resulting in great cleanliness.

3.2.3 Classification

In the construction environment are present various classifications of CSTCBs. One of the most significative is that proposed into [18] (“REP®beam” is the commercial name used by “Tecnostrutture S.r.l.” for the composite steel trussed concrete beam). In the citated text, is made a distinction between different types of beams:

1. CSTC beam NOR
2. CSTC beam TR
3. CSTC beam DIS
4. CSTC beam CLS
5. CSTC beam PRE

In the list, beams 1, 2, 3 are all made with a bottom steel plate with dissimilar configurations of the main steel truss, but in particular the main difference is that NOR and TR beams need an additional type of truss at the nodes to create effective connections, while DIS are beams that continue beyond the node by extending into adjacent spans for the length required to absorb the interlocking moments.

Beams 4, 5 are made instead with a bottom chord in concrete. Additionally exist other types of CSTCBs such as “REI S beam”, “REI 5 beam”, “ISO beam” and “LC beam” that are not been analysed in this study due to their really specific field of application but are worth to be mentioned.

CSTC beam NOR

The self-supporting CSCTB NOR permits the realisation of large-span constructions. Its framework, made entirely of plain S355 steel in compliance with the UNI EN 10025-2 standard, consists of the following elements (Figure 79):

1. A lower chord, formed by a plate and optionally reinforced with additional welded iron components.
2. An upper chord composed of at least one pair of steel sections.
3. A connection web, either single or double, welded to both the upper and lower chords.
4. Support terminals precisely dimensioned to provide secure anchoring and resist sliding forces.

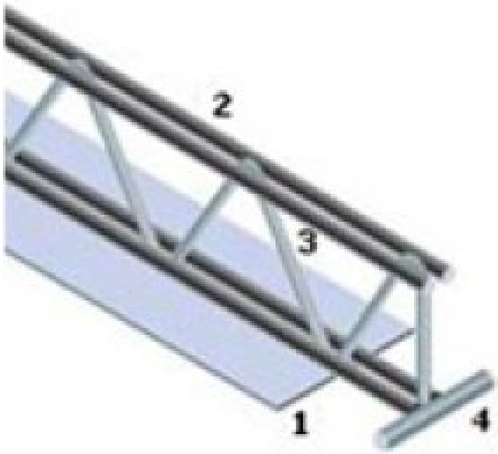

Figure 79 Steel truss of NOR beam.

The steel plate serves dual purposes: it provides support for the slab and functions as a formwork for the completion casting. The CSTC NOR Beam is self-sustaining for its own weight and accommodates the slab's load even during the initial phase of construction and rest on pillars using some patented connections provided at their ends. These act as a sort of additional reinforcement at the nodes (Figure 80) that provide the continuity of the elements. These trusses are produced in general in different dimensions in order to be simply put on site in the correct position before the casting without any other additional operation. Moreover, these hybrid beams can be supplied with particular shapes of the bottom plate and with openings for the passage of systems according to project requirements. Visible parts can be made of COR-TEN steel or treated with rust inhibitor.

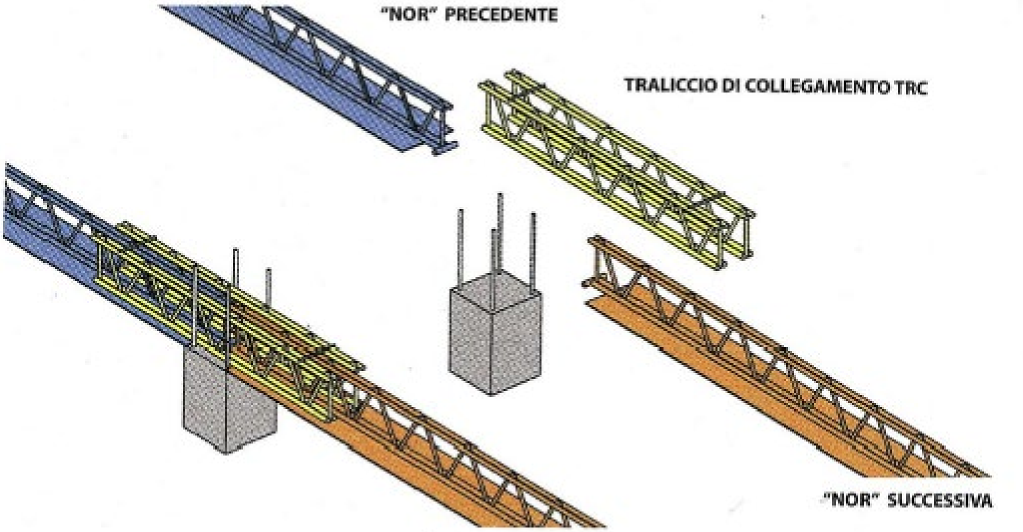

Figure 80 Additional steel truss for the connection at nodes between two CSTC beams NOR.

<u>CSTC beam TR</u>

It is suitable for use with slabs, such as predalles, as it does not require any additional formwork. The metal structure, made entirely of plain steel S355 in accordance with UNI EN 10025-2, consists of one or more trusses (Figure 81) made up of:

1. Upper chord, formed of at least one pair of steel profiles;
2. Lower chord, formed of at least one pair of steel profiles;
3. Connecting web, single or double, welded to the upper and lower chords;
4. Lower steel linking elements.

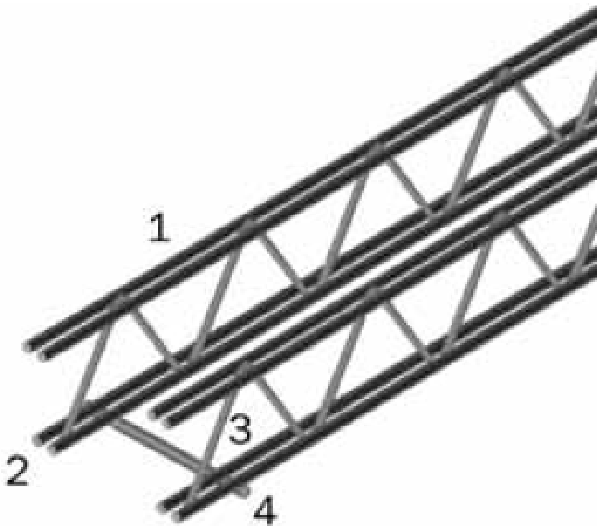

Figure 81 Steel truss of TR beam.

In the same way of the previously described beams, TR beams need the compenetration of truss elements at the nodes (connections) in order to guarantee the structural continuity and absorb bending moments (Figure 82).

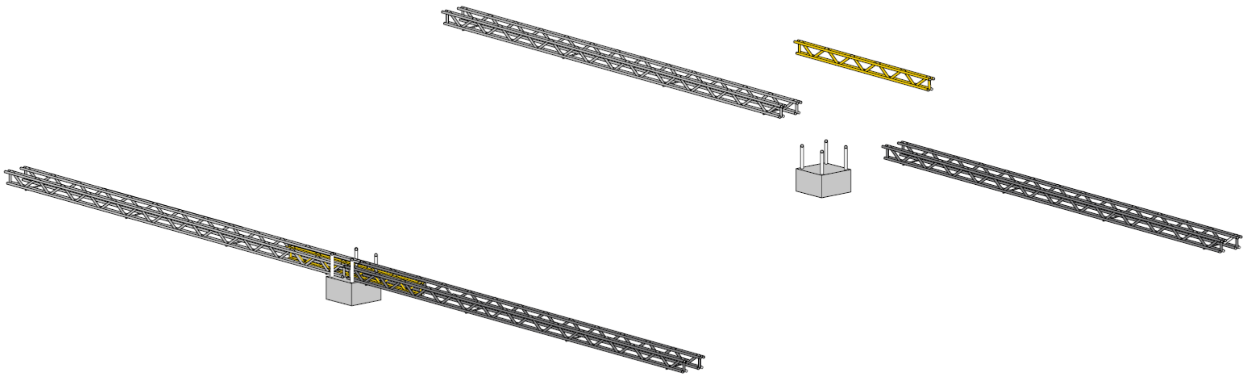

Figure 82 Additional steel truss for the connection at nodes between two CSTC beams NOR.

The CSCT TR Beam, which is industrially produced by coupling standard profiles that complement each other, has a specific load-bearing capacity that is defined according to the height and section of the profiles. Depending on the design stresses, the beam has a variable number of trusses. These can be connected by lower steel linking elements that function as stabilisers during casting and as spacers from the formwork.

The main advantage of the use of this type of beams is its high rigidity and high percentage of reinforcement. The consequences of that are the possibility to increase the span between the structural elements and the reduction of the width of beams (and so of the weight to be transferred at the foundation).

CSTC beam DIS

The name of this beam “DIS” derives from the Italian “trave disassata”: it is a beam made with a single truss that is not aligned with the centreline of the beam. This technology allows for the overlapping of the trusses of two adjacent beams to provide the continuity at the node (Figure 84). In this way, they don’t need any additional truss at the level of the connection. This aspect ensures an easier and faster assembly of components. Anyway, the additional reinforcement can be added in seismic zones in order to absorb in a better way the stressed at bearings.

The metal structure, made entirely of plain steel from S355 in accordance with standard UNI EN 10025-2, consists of (Figure 83):

1. Upper chord formed of at least one pair of steel profiles;
2. Lower chord formed of at least one pair of steel profiles and a non-coaxial plate welded to the truss;

3. Connection web welded to the upper and lower chords.

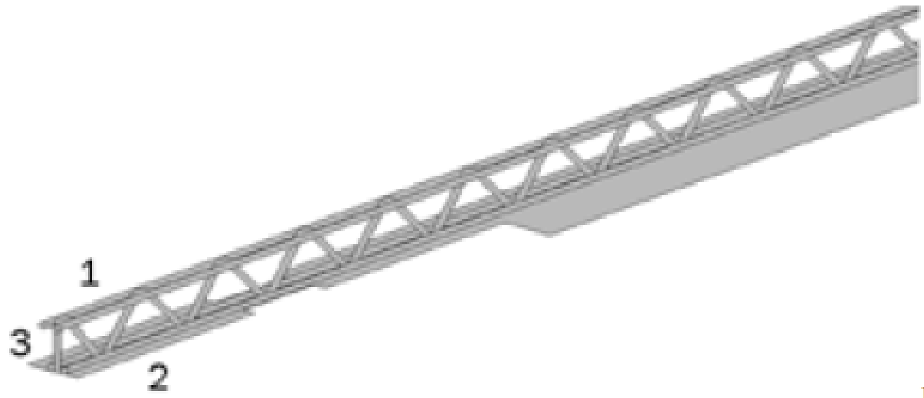

Figure 83 Steel truss of TR beam.

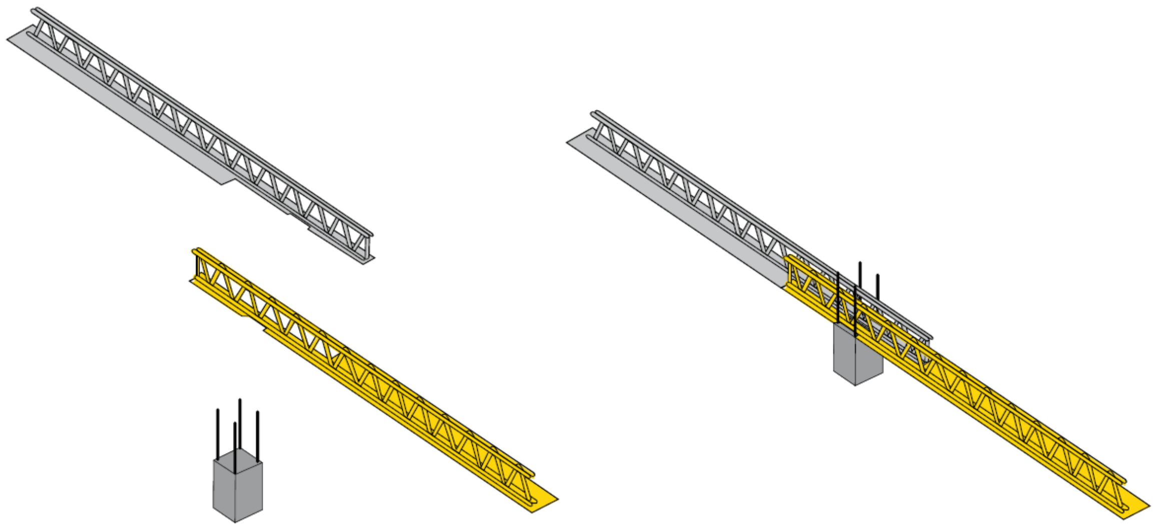

Figure 84 Overlapping of the trusses for the connection at nodes between two CSTC beams DIS.

CSTC beam CLS

This self-supporting beam is made in general by two main components (Figure 85):

- A steel reinforcement made up of one or more smooth steel trusses S355 welded in fabric with continuous wire under protective gas;
- A bottom chord made in reinforced precast concrete base, usually C28/35, sized to support the loads of the sabs.

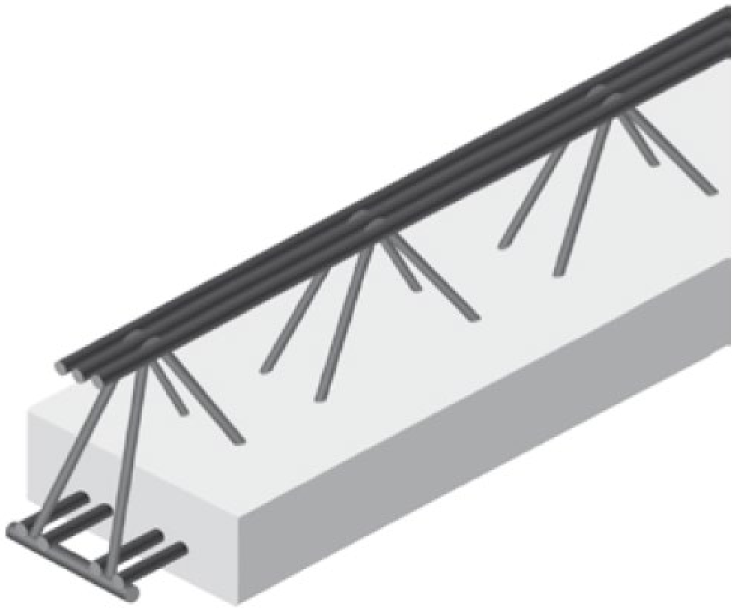

Figure 85 Render of a CSTC beam CLS.

This technology is ideal for constructing structures demanding exceptional performance under heavy loads or extended spans, and is particularly recommended for hollow core slabs or standard slabs.

The connection with the pillars is made generally with the use of special components, called hammers, situated at the extremities of the beams. Additionally, is placed an integrative reinforcement at the node to guarantee structural continuity.

In particular cases, steel bars with improved adherence are supplied at beam-pillar nodes. In case of exposed steel parts, they can be made in COR-TEN steel to improve the resistance to corrosion processes.

<u>CSTC beam PRE</u>

This type of beam is similar to the previously described "CSTC beam CLS". The main difference can be found directly in the name "CSTC beam PRE": in fact, "PRE" stands for prestressed concrete.

This hybrid beam is so composed by (Figure 86):

- A steel reinforcement made up of one or more smooth steel trusses S355 welded in fabric with continuous wire under protective gas;
- A bottom chord made in precast concrete base, usually 32/40, equipped with some prestressed tendons.

The presence of prestressing is really important because guarantees the self-supporting of the base slab and contributes to the overall load-bearing capacity of the beam, making it possible to reduce the dimensions of the truss.

The connection with the pillars is the same of the “CSTC beam CLS”.

Figure 86 Render of a CSTC beam PRE.

3.3 STRUCTURAL BEHAVIOUR OF CSTCBs

3.3.1 Regulations

For over 40 years, a wide range of truss designs have been developed, yet there has been a noticeable lack of technical and operational guidelines from any external authorities to assist in their design and implementation.

The path toward regulations of CSTCBs started formally by “Gruppo di Lavoro Assoprem (Associazione nazionale produttori di travi PREM)” in 2006 and, after years of research and sperimentation, concluded with the publication in 2009 of “Raccomandazioni per la progettazione e l’esecuzione di travi prefabbricate reticolari miste” [23]. During these years came into effect NTC 2008, that at § 4.6 refers to the possibility of utilization of CSTCBs.

In 2011, the CSLLPP (“Consiglio Superiore dei Lavori Pubblici”) publishes on its website the “Guidelines for the use of steel trusses embedded in the collaborating concrete casting and procedures for the issue of the authorisation for use” [24] in application of § 4.6 of the NTCs which include such beams among the “non-traditional materials or materials not covered by these standards”. These identify three categories within which to place each structural type:

A. composite steel-concrete structures (with all and only rules from mixed structures);
B. normal or prestressed reinforced concrete structures (with all and only rules from reinforced concrete);
C. structures that do not fall within the principles, definitions, calculation models and materials of the two categories listed above".

In particular Category A includes beams made only from structural steel. Such prefabricated elements follow the requirements for steel-concrete composite structures. The prefabricated bottom chord, if present, may be made of steel or reinforced concrete. The bearing capacity in phase I is addressed only to the steel truss, in particular due to the definition of composite structures, shear resistance is fully taken by structural steel.

Category B implies that the bearing capacity of phase II is given only by reinforced concrete, following the specific regulations of NTC. In this case structural steel (steel for profiles) has not to be considered for the bearing capacity of the whole section.

Category C are instead hybrid structures that represent an innovation in the field of structural engineering, for which the regulations have not yet provided specific design rules.

Mostly of the CSTCBs present in the commerce can be inserted in the definition of Category C, and so, as foreseen by Ministerial Procedures, they need a deep sperimentation and research due to the current lack of formulations for the main verifications. These includes experimental tests in support to the definition of reliable calculation models.

Due to this approach a lot of tests and studies have been made in the last years, in order to validate models and propose simplificative formulations for the main verifications. In the next chapters will be faced in detail these aspects.

3.3.2 Introduction and verifications

Structure idealised with CSTCBs have a different behaviour respect to the traditional structural types. This is mainly because their structural behaviour and static scheme evolve during the construction phase. [18] emphasises two different operating phases (Figure 87):

- Phase I: This lasts until the concrete casting reaches its intended design strength. During this phase the self-bearing beam behaves as a steel truss: depending on the construction choice, self-supporting can be complete or partial (where a portion of the floor weight is distributed to a supplementary propping system). The specific static scheme associated to this phase is in general a simply supported trussed scheme resting on vertical structures, as in Figure 88. The calculation span of the beams matches the distance between the supporting hammer points. Only vertical loads aligned with the axis of the hammer, supporting the beams, transfer the load to the walls and columns.
  During this phase, there are no tension stresses within the concrete. All the loads, among which the self-weight of the beams, the concrete casting, and the precast slabs, are exclusively supported by the welded steel components of the beams. This prevents cracking or viscosity effects. Any supplementary reinforcement, such as bars or integrative connections, also remains unstressed during this stage.
- Phase II: This phase marks a critical stage where the concrete casting, having matured, achieve full cooperation with the welded metal parts, the in-situ additional reinforcement, and the surrounding structure. At this point the structure becomes a composite one where concrete and steel experience the same deformations, ensured by the penetration of concrete between the links of the lattice steel structure. In this stage, the concrete casting has hardened, the beam is connected to the adjacent beams and columns and the static scheme becomes that of a continuous beam with intermediate bearings and clamped endings, as in Figure 89.
  The structure now accomplishes its monolithic character, allowing for the transmission of bending moments at the nodes between beams and columns or walls. The loads experienced in this phase apply stresses on both the steel and concrete components. Long-duration loads, such as permanent carried loads and quasi-permanent portions of accidental loads, induce viscosity effects in the concrete. Conversely, short-duration loads, such as instantaneous accidental loads, wind, or seismic forces, do not result in rheological behaviours.

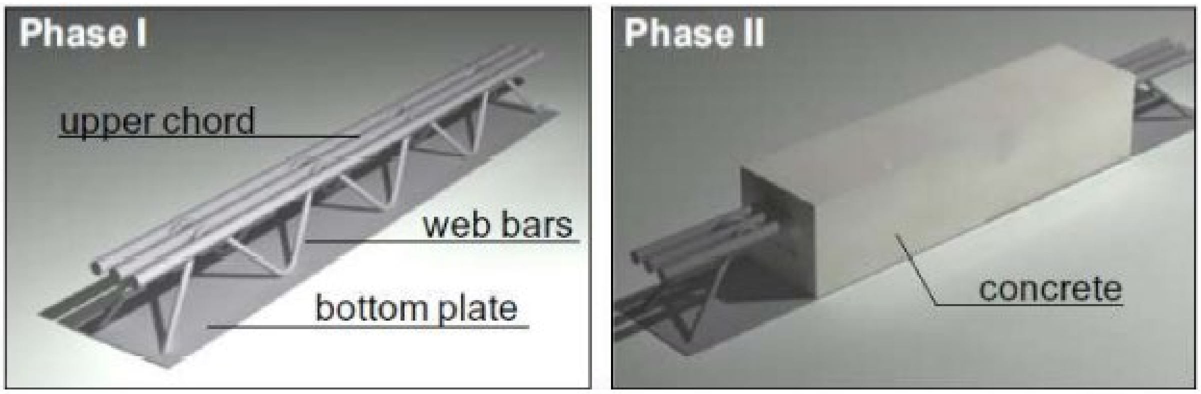

Figure 87 Beam structural schemes in operating Phases I and II.

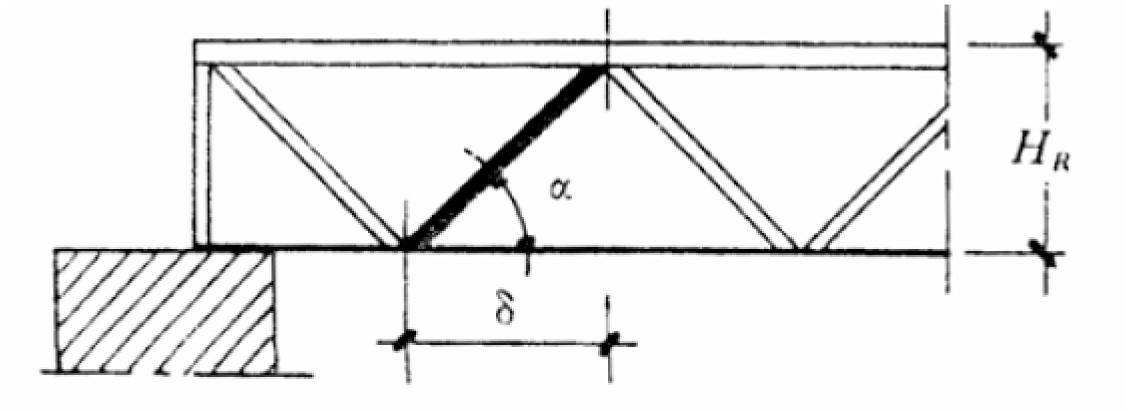

Figure 88 Simply supported scheme of the beam in phase I.

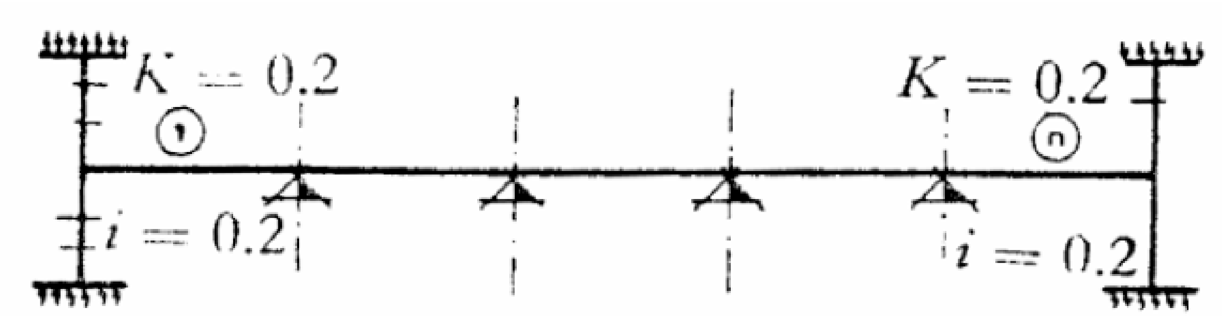

Figure 89 Continuous scheme of the beam in phase II.

It is essential to account for the distinction between the first and second phases during the design process, especially when calculating stresses and verifying stress states. This differentiation is particularly relevant in Serviceability Limit States Verifications: verifications in operational conditions, cracking, and deformability. However, this distinction becomes almost irrelevant when assessing failure modes at the Ultimate Limit State.

Stresses at the different stages

The CSTCB system requires loads to be subdivided into those acting on the steel truss only (without the collaboration of the concrete) considered in simply supported configuration, and those acting in the second phase on the complete structure. The main distinction can be made between:

1. Loads in phase I:
   - weight of steel truss;
   - weight of the concrete casting ($2500\ kg/m^3$);
   - weight of the influence area of the slab;
   - additional loads, both concentrated and distributed, including any loads from structural elements permanently resting on the beam or temporarily placed on it in the 1st phase.
2. Permanent Loads in phase II:
   - permanent loads resulting from the 1st phase;
   - incidence of partitions;
   - weight of screed, pavement and plaster;
   - weight of fixed furniture;
   - additional fixed loads, such as loads from the façade.
3. Accidental Loads in phase II:
   - Accidental loads from external factors, such as explosions.

Main verifications in phase I

In phase I the hybrid beam is dimensioned as a steel truss only, considering isostatic scheme. So, at this stage it is considered self-supporting and will carry all the loads previously defined in “Loads in phase I”. The maximum bending moment is in midspan:

$$M_{max} = \frac{gL^2}{8} \tag{14}$$

The maximum shear is at the bearings:

$$V_{max} = \frac{gL}{2} \tag{15}$$

where *g* is the design load associated to phase I and *L* is the span of the beam.

Since in CSTC Beams the 1st phase loads have no effect in the ending sections (level of connections), there is the transfer of a share of the bending moment from the edge section to midspan. Therefore, compared to ordinary reinforced concrete beams, less bending stress occurs in the beam-to-column connection.

Due to the isostatic scheme and so due to the positive bending moment, the upper rebars (top chord) of the steel truss are subjected to compression. In steel structures compression is particularly relevant, and so those must be verified against buckling in the vertical plane (Figure 90) and in the horizontal one, following the verifications related to “steel structures” in NTC2018. Additionally has to be verified also the stability of the first diagonal element of the truss in compression (Figure 91).

For the behaviour in shear is important to underline that the whole shear action V_{max} is taken only by the web of the truss and must be lower or equal to its shear resistance.

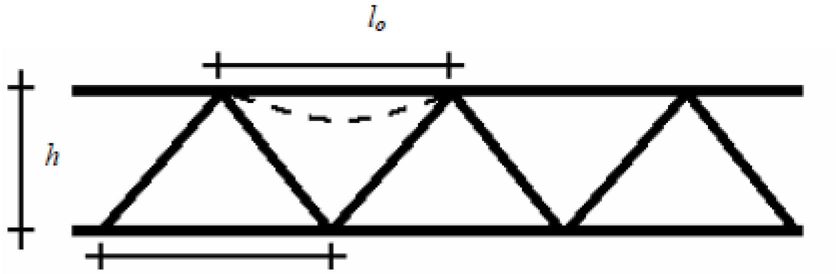

Figure 90 Buckling verification of the top chord in the vertical plane highlighting the inflection length l_0 between two diagonal elements.

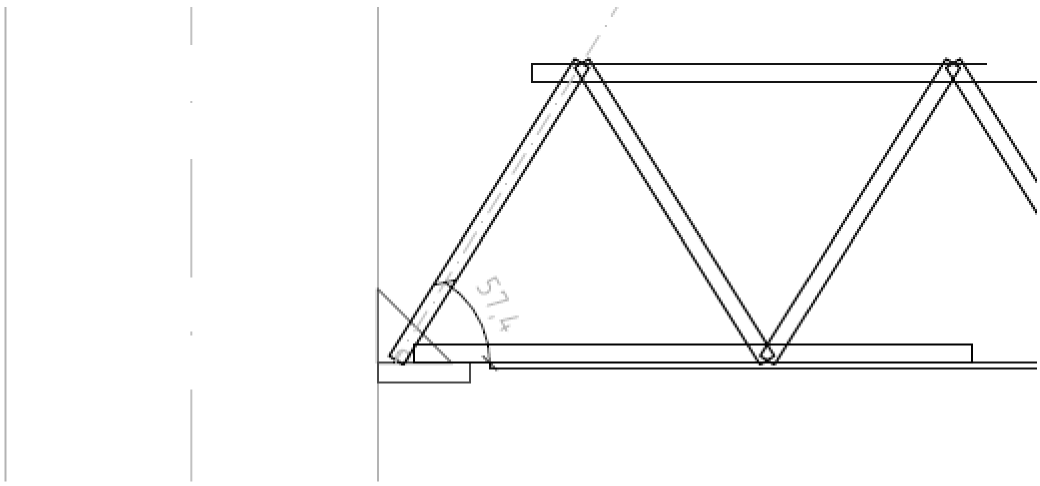

Figure 91 Stability verification of the first diagonal element near to the bearing.

Main verifications in phase II

In phase II, after hardening and curing of concrete, is achieved the full collaboration between concrete and structural steel and the beam now can be considered as a whole with a continuous static scheme on elastic bearings.

The verifications in bending of the entire hybrid beam take into account the stress envelopes that provide the highest characteristic stresses in the considered section (respectively in midspan and in the sections at the bearings). The resisting moment is obtained after finding the position of the neutral axis and must be higher or equal to the maximum acting moment.

In recent years has been assessed that also at this stage the shear stress is entirely absorbed by the web of the steel truss, providing therefore that the adherence between the casted concrete and the bottom plate of the beam is irrelevant. Due to this important consideration the shear verifications must be performed only at the level of the steel web of the truss and considering the maximum value of the shear action (at the beam-column connection).

The bottom chord of the truss is the tensile-resistant element of the beam that must be dimensioned in function of the effects of the 1st phase combined with the stresses due to the maximum bending value acting in the 2nd phase.

Verification of the deformations

Deformations caused by 1st phase loads are evaluated independently from those resulting from fixed loads and accidental overloads. The assessment of 1st phase loads focuses only on the properties of the metal component, while both instantaneous and time-dependent elastic effects are taken into account for the final structure.

When calculating deflection, it is essential to consider the different stages in the behaviour of the CSTC beam. This includes calculating the deflection of the steel truss alone under initial loads (loads of phase I), the deflection of the final hybrid beam accounting for displacements due to permanent loads and due to overloads.

3.3.3 Structural performance

CSCT beams are widely employed in industrial buildings and in recent years is growing the interest from the scientific community in investigating their performance in residential buildings, emphasizing both static and seismic responses. The technical studies [25] indicate that CSCTBs operate under different mechanical principles compared to RC or steel-concrete composite beams. Consequently, substantial efforts have been made over the past decades to establish standardized design guidelines in national and international codes by thoroughly understanding their primary resistance mechanisms. More recent research has therefore focused on gaining deeper insights into the unique structural behaviour of CSTCBs, particularly in relation to flexure and shear.

In the following paragraph will be provided an analysis on the behaviour of hybrid beams in phase I and II though the analysis of some tests from [25], oriented to obtain simplified valid analytical models and formulations for the verifications. The analysis focus on:

1. Assessment of the behaviour in phase I:
    - Push out response on bare specimen,
2. Assessment of the behaviour in phase II:
    - Bending and shear capacity.

3.3.3.1 Assessment of the behaviour in phase I

The behaviour of the beam during Phase I relates to the transient period of the construction process when the steel truss remains exposed, before the concrete casting. Regarding structural behaviour during this phase, one of the primary issues is the buckling phenomenon affecting either the compressed web bars or the upper chord of the steel truss. Three distinct buckling modes have been identified:

1. Instability of the upper chord, characterized by a buckling length affecting a single rod. In this case can be applied the classical instability criteria where buckling length is determined by either the spacing between transverse restraints on the upper chord or the rod length of web trusses.
2. Instability of the compressed upper chord, where the buckling length involves multiple mesh sections of the truss. Generally happens when transverse restraints are not sufficiently rigid to prevent displacement of nodes within the upper chord.
3. Coupled flexural-torsional instability within the steel truss. It could affect the entire beam.

In the last two cases, it is notable that the critical load value is considerably lower than the Eulerian critical load for individual rods.

Push out response on bare specimen

To better understand the behaviour of Hybrid Beams in Phase I, [26] conducted monotonic push-out tests under displacement control. These tests were performed on specimens constructed following the scheme illustrated in Figure 93. Two distinct tests were carried out on specimens S1 and S2. Specimen S1 consisted of two truss sections symmetrically connected along the middle lines of the bottom plate using an 8 mm thick steel plate placed perpendicular to the plates of the steel trusses. The truss beam in S1 incorporated an upper chord composed of three coupled rebars, each with a diameter of 16 mm, and a bottom chord made of a 5 mm S355 steel plate. The web bars, constructed from 12 mm B450C rebars, were arranged in a reverse V-shaped configuration, giving the truss beam an overall depth of 210 mm. To facilitate load application, an 8 mm thick plate was mounted orthogonal to the specimen’s axis at its top and was appropriately stiffened for compatibility with the testing machine. Specimen S2 differed from S1 through the addition of two longitudinal plates, each measuring 40 mm wide and 5 mm thick. These plates were welded to the web bars to decrease their slenderness and mitigate the risk of buckling. Both specimens displayed comparable peak resistances, measured at 328 kN for Specimen S1 and 333 kN for Specimen S2.

[27] proposed a simplified mechanical model based on the static system of Figure 92 to understand the experimental results.

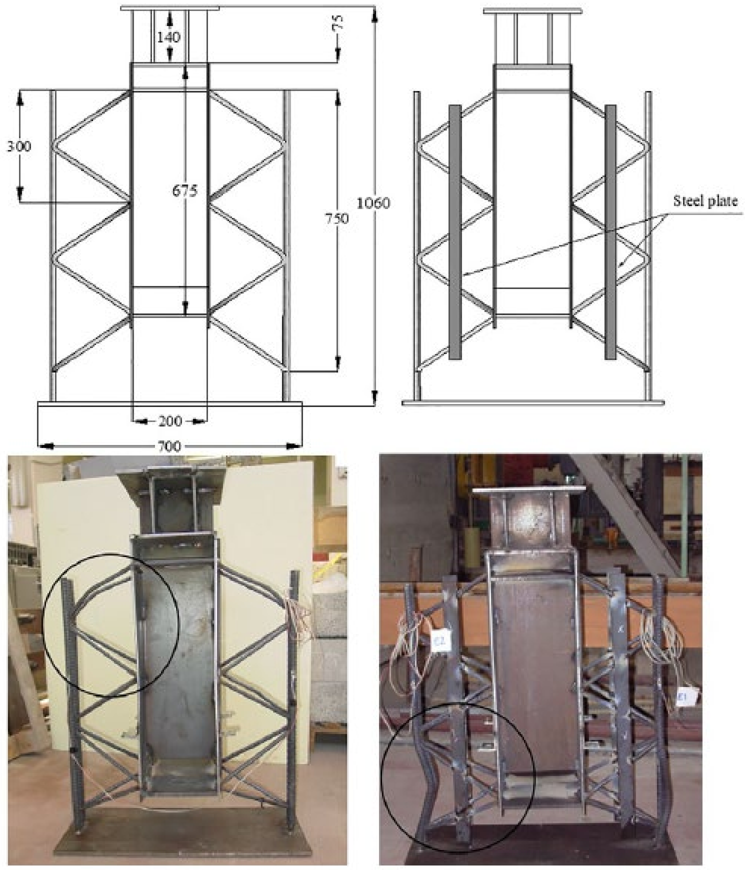

Figure 93 Specimens for push-out test in Phase I: specimen S1 without vertical restraints (left) and specimen S2 with stiffeners (right).

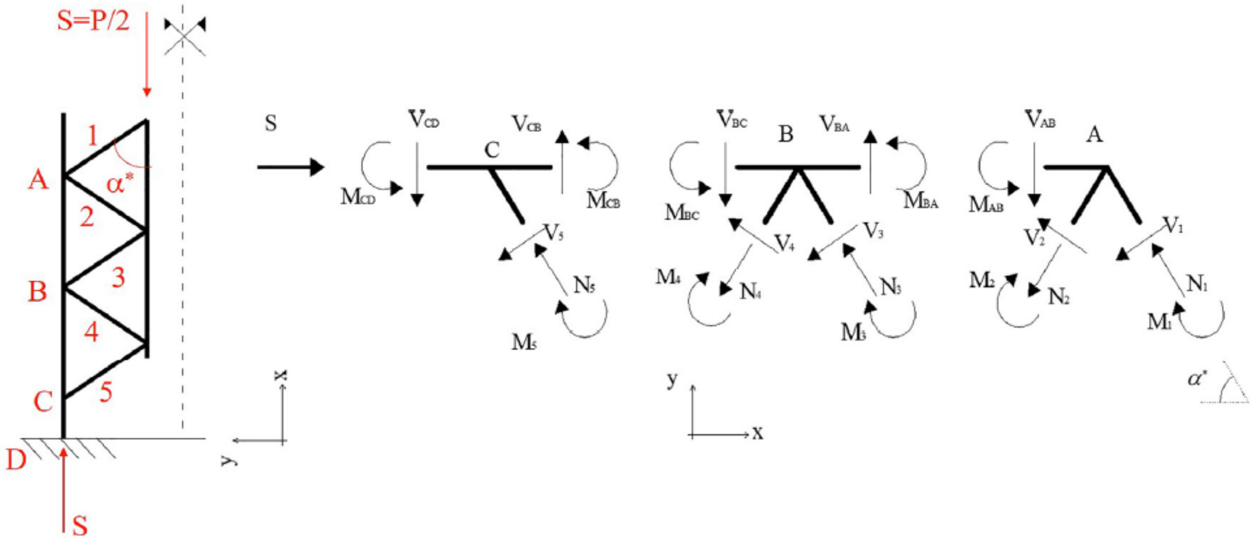

Figure 92 Simplified mechanical model for pushout resistance in Phase I.

Considering the symbols in figure 8 is possible to write the equilibrium equations:

$$(N_1 - N_2)\sin\alpha - \frac{2}{l_\omega}(M_1 - M_2)\cos\alpha - \frac{1}{2l_c}(2(M_1 + M_2) + M_3 + M_4) = 0 \tag{16}$$

$$(N_3 - N_4)\sin\alpha - \frac{2(M_3 - M_4)\cos\alpha}{l_\omega} - \frac{3M_5}{l_c} + \frac{\rho_{CB}}{\rho_{CB} + \rho_{CD}} + \\ + \frac{1}{2l_c}(2(M_1 + M_2) + M_3 + M_4) = 0 \tag{17}$$

where ρ_{CB} and ρ_{CD} are the flexural stiffness of the rods named CB an CD, while l_ω and l_C are respectively the length of the diagonal bar and the upper chord.

The maximum external force S that can be applied is determined by the Author [27] with an analysis based on the lower bound theorem of plasticity and on the allowable plastic and buckling conditions:

$$M_u = \frac{d^3}{6} f_y \left(1 - \frac{N_u^2}{N_y^2}\right) \tag{18}$$

$$\frac{N}{N^*} + \frac{M}{M^*} \leq 1 \tag{19}$$

Where d is the diameter of the rod, M_u and N_u are the ultimate bending moment and axial force, while N_y and f_y are the yielding axial force and tensile strength. Considering Eq. (19), $N^* = N_y$ in tension, while $N^* = N_b$ in compression, and M^* represents the yielding or plastic bending moment in the case of yielding domain or plastic domain, respectively.

So finally, is obtained:

$$S = \left(\sum_{i=1}^{5} N_i\right)\cos\alpha + \frac{2}{l_\omega}\left(\sum_{i=1}^{5} M_i\right)\sin\alpha \tag{20}$$

The analytical estimation of the maximum load, P, is found from the peak value of S while accounting for the interaction between the two trusses forming the beam. This resulted in an estimated value of P = 326.64 kN, yielding a predicted-to-experimental ratio of 0.996. Additionally, the failure mechanism of specimen S2 with stiffening plates was modelled differently, involving buckling of the top chord. Simplified evaluation, based on the reduced axial strength of the top chord (N_b = 66.5 kN) and considering its composition of three rebars per truss in the specimen, allowed for an assessment of the maximum load as $P = 6\,N_b = 399\,kN$. However, this approach led to an overestimation of approximately 21%.

3.3.3.2 Assessment of the behaviour in phase II

Several investigations have been made to assess the mechanical behaviour of CSTCBs during phase II, so after concrete casting and hardening. The most relevant topic addressed in this paragraph is about the interpretation and the evaluation of flexural and shear resistance.

<u>**Bending and shear resistance**</u>

The literature includes numerous studies focused on analysing the behaviour of CSTCBs under flexural and shear stress. These investigations typically begin with experimental testing, followed by interpretation through analytical and numerical modelling. This section provides the key considerations conducted during phase II by L. Tesser and R. Scotta [19].

L. Tesser and R. Scotta conducted experiments on hybrid beams presenting a precast base enveloping the bottom chord of the steel truss. Their testing framework was designed to subject each specimen to two distinct failure modes: flexural failure and shear failure, as illustrated in Figure 94. In total 12 beams have been subjected to the tests.

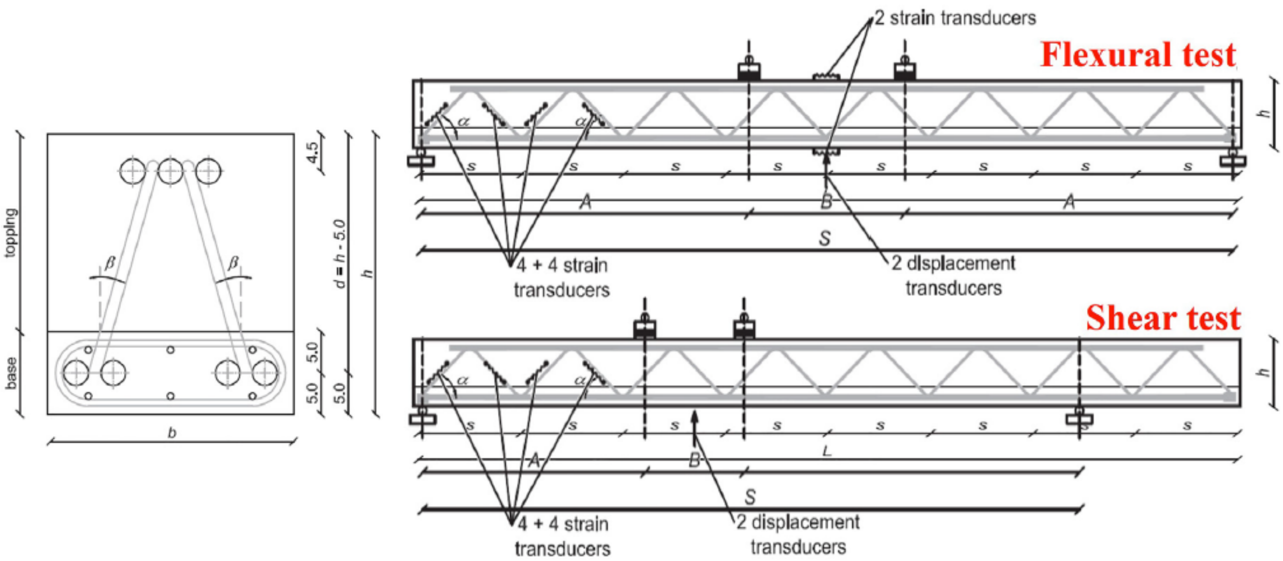

Figure 94 Specimens tested by Tesser and Scotta (dimensions in cm).

Table 11 outlines the geometrical attributes of the beams, with the symbols representing the following parameters: *L* denotes the beam length, *b* specifies the width of the beam section, *h* represents the depth of the beam section, *s* indicates the spacing between successive bent web bars, *α* defines the angle between a bent-up web bar and the member's longitudinal axis, and *β* refers to the angle between the direction of shear force and a web bar within the beam's cross-sectional plane. Additionally, each concrete base has a depth of 0.10 m and includes six longitudinal bars ϕ8, accompanied by rectangular stirrups ϕ8 arranged at 0.15 m intervals. The prescribed material properties are normal strength concrete of class C50/60 with a maximum aggregate size of 15 mm and hot rolled structural steel S355JR.

ID	Beam geometry			Steel truss					
	L (m)	b (m)	h (m)	Top chord	Bottom chord	Web	s (m)	α (°)	β (°)
B1	6.00	0.25	0.24	3∅28	4∅28	2∅14	0.40	40	18
B2	6.00	0.30	0.24	3∅30	4∅30	2∅14	0.40	40	25
B3	6.00	0.40	0.24	3∅28	4∅28	2∅14	0.40	40	39
B4	6.00	0.25	0.34	3∅30	4∅30	2∅14	0.40	53	11
B5	6.00	0.30	0.34	3∅30	4∅30	2∅14	0.40	53	16
B6	6.00	0.40	0.34	3∅28	4∅28	2∅14	0.40	53	26
B7	6.00	0.25	0.44	3∅26	4∅28	2∅14	0.40	62	8
B8	6.00	0.30	0.44	3∅30	4∅30	2∅14	0.40	62	12
B9	6.00	0.40	0.44	3∅30	4∅30	2∅14	0.40	62	19
B10	6.44	0.25	0.54	3∅30	4∅30	2∅16	0.46	65	6
B11	6.44	0.30	0.54	3∅30	4∅30	2∅14	0.46	65	9
B12	6.44	0.40	0.54	3∅30	4∅30	2∅14	0.46	65	15

Table 11 Geometrical properties of the 12 specimens.

The experimental setup is depicted in Figure 96. The load application is controlled by a hydraulic jack mounted on a steel frame securely anchored to the laboratory floor. The jack is operated using a hydraulic control unit that impose the required displacements. A load cell positioned between the jack and a distributor beam measure the applied force with greater accuracy. The distributor beam is simply supported and centrally loaded, enabling the specimen to be subjected to two identical forces, whose positions are variable. The steel beam transfers the load to the specimen via two low rails, which are oriented along the longitudinal direction of the specimen and have a size of 0.12 m. To eliminate horizontal constraint effects, three thick neoprene layers are placed between the rail base and the specimen. The specimen itself is held in place by an additional pair of steel low rails mounted on reinforced steel pedestals. The cumulative weight of the superstructure applying the load is approximately 4.1 kN. The specimen's static configuration corresponds to a simply supported beam subjected to two-point loads. Throughout the testing process, both local and global displacements have been recorded. For displacement measurements, strain transducers with a measurement base of 100 mm are employed.

The crack patterns identified during the flexural tests are marked by cracks wider than 0.5 mm, predominantly located in the central sections of the beams (Figure 95). These cracks follow a sub-vertical trajectory, extending from the concrete base to the concrete topping. All beams demonstrate yielding in the lower chord regardless of the variations in section sizes and longitudinal reinforcement ratios. Furthermore, specimens B3 and B6 exhibit concrete crushing in the upper section between the two load application points.

The shear tests reveal a distinct crack pattern characterized by at least one diagonal fissure connecting the support to the nearest loading axis within the upper concrete section. This crack gradually widened, extending toward the upper section edge on one end and the interface between concrete casts on the other. Following this progression, cracks in the interface area move horizontally toward the nearest support, with the exception of beam B8.

At peak stress levels, two distinct phenomena emerge, either independently or simultaneously. One occurrence involves the widening of the interface crack as it propagates toward the support, frequently leading to detachment of the concrete base near the termination of the lower chord. The other phenomenon sees interface cracks extending diagonally in the concrete base, generally offset from the concrete topping. Importantly, in all instances, the cracks formed during shear tests remained separate from those observed during flexural testing.

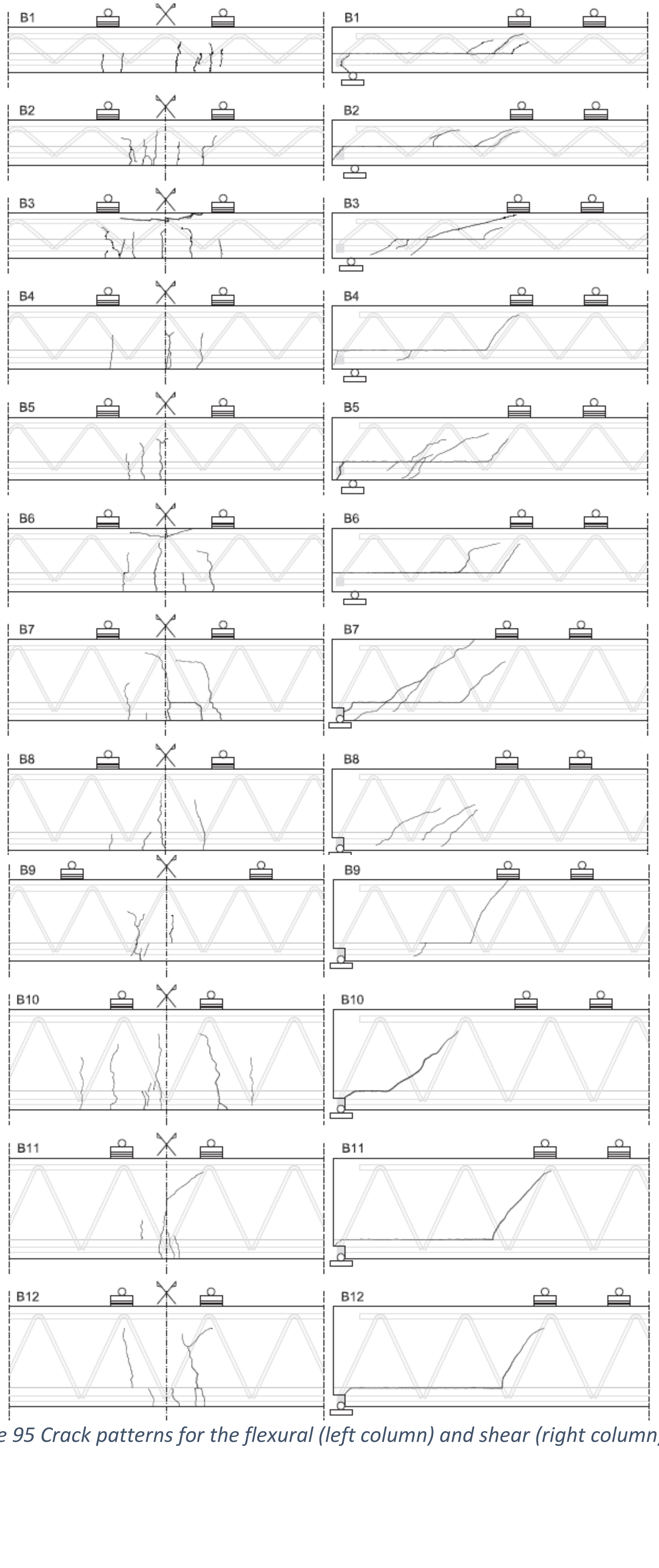

Figure 95 Crack patterns for the flexural (left column) and shear (right column) tests.

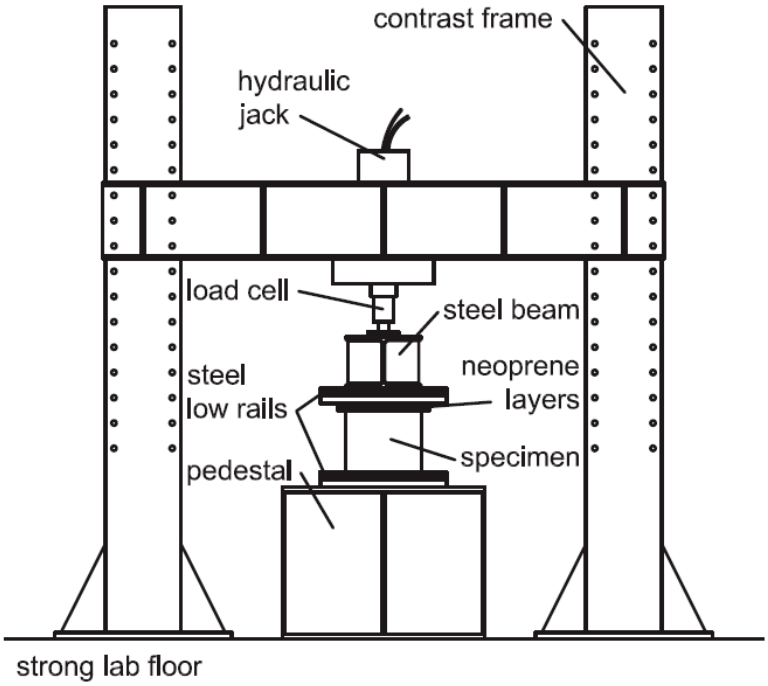

Figure 96 Test set up.

As next step, the Authors try to make an analytical interpretation of the results.

For the assessment of the bending capacity, they followed the formulation of Eurocode 2:

$$M_{rd} = f_y A_s d \left(1 - \frac{a}{d}\frac{x}{d}\right) \tag{21}$$

$$\frac{x}{d} = \frac{\frac{\sigma_s}{f_y}\,A_s - \frac{\sigma_s'}{f_y}\,A_s'}{\beta_r f_c' b_{eff} d} \tag{22}$$

where A_s and A_s' are the tensile and compressive steel reinforcement, respectively, while σ_s and σ_s' are the corresponding stresses; d is the effective depth of the beam, f_c' the concrete cylindrical compressive strength, b_{eff} the effective compressive concrete width, and β_r a coefficient for the integration of the stress distribution over the section (equal to 0.8 if the stress block simplification is assumed).

For the evaluation of the shear capacity the authors proposed various models from the codes, here is reported the formulation from Model Code 2010. The shear strength can be evaluated as the sum of concrete and steel contributions:

$$V_{rd} = V_{c,rd} + V_{s,rd} \tag{23}$$

Where:

$$V_{c,rd} = k_v\, bz\sqrt{f_c'} \tag{24}$$

$$V_{s,rd} = A_{sw}\ f_{yw}\ (\cot\theta + \cot\alpha)\sin\alpha\ \cos\beta\ \frac{z}{s} \tag{25}$$

In these equations k_v is a coefficient depending on the concrete compressive strength, in the range 0.145 - 0.178 for the tested beams; z is the inner lever arm of the section; A_{sw} and f_{yw} are the area and yield strength of the transversal reinforcement, respectively; θ indicates the inclination of the compression strut to the beam axis.

The comparison between experimental and theoretical results is reported in Table 12 in terms of the errors given by the validation. More in detail, for each specimen, for the flexural and shear capacity, the error is measured as:

$$Error\ flex\ = \frac{M_{rd} - M_{exp}}{M_{exp}}\ \% \tag{26}$$

$$Error\ shear\ = \frac{V_{rd} - V_{exp}}{V_{exp}}\ \% \tag{27}$$

ID	$P_{max,flex}$ [kN]	$P_{max,shear}$ [kN]	Error flex [%]	Error shear [%]
B1	111	149	−9.7	24
B2	150	192	−21.4	4
B3	143	263	−21.4	−22
B4	211	293	−6.4	2
B5	208	382	−7.7	−18
B6	193	280	−15.2	15
B7	276	490	−17.5	−22
B8	313	496	−9.9	−16
B9	303	501	−4.2	−10
B10	332	457	−10.4	5
B11	326	376	−6.8	17
B12	328	381	−9.2	23
Average			−11.6	0
St. Dev.			5.8	17

Table 12 Flexural and shear resistance of the specimens tested by Tesser and Scotta.

3.3.4 Structural effect of creep

When a concrete specimen is subjected to a load, its behaviour exhibits both immediate and time-dependent responses. Under a constant load, the deformation of the specimen progressively increases over time, potentially becoming several times greater than its initial deformation. To fulfil serviceability requirements in structural design, it is essential to make precise predictions regarding the instantaneous and time-dependent deformation of concrete structures. If temperature and stress conditions are stable, the gradual increase in strain over time is attributed to the effects of creep and shrinkage.

Creep refers to the time-dependent deformation of concrete under a constant load. When a concrete member is loaded and held under load for an extended period, it experiences some deformations even if the load is constant.

In Composite steel trussed concrete beams, the effect of creep is really important because significantly affects the stress redistribution due to the change of the static scheme over the time. In fact, as seen previously the structural scheme evolves from phase I to phase II (Table 13):

- Phase I: isostatic simply supported scheme, before the hardening of the casted concrete in situ.
- Phase II: continuous scheme after the casting and hardening of concrete.

Time	Event	Structural Scheme	Stress Response
t = 0	Steel truss placed	Isostatic	Steel carries initial loads
t = t_1	Concrete is cast	Isostatic	Wet concrete adds self-weight; steel carries it
t = t_2	Concrete hardens	Statically determinate (still isostatic)	Composite action begins
t = t_3	Connections are activated (continuous behaviour begins)	Statically indeterminate	Internal forces redistribute due to compatibility and creep

Table 13 Brief summary of what happens into a CSTCB from the assemble on site to the hardening of concrete.

At any time t, the total concrete strain ε(t) in an uncracked, uniaxially-loaded specimen consists of a number of components that include the instantaneous strain ε_e(t), creep strain ε_{cc}(t), shrinkage strain ε_{sh}(t) and temperature strain ε_T(t) [28].

While this assumption is not entirely precise, it is a generally accepted practice to treat these four components as independent, allowing them to be calculated separately and then combined to determine the total strain. When analysing the in-service behaviour of a concrete structure under constant temperature conditions, the concrete strain at a specific point is typically expressed as the sum of the instantaneous, creep, and shrinkage components:

$$\varepsilon(t) = \varepsilon_e(t) + \varepsilon_{cc}(t) + \varepsilon_{sh}(t) \tag{28}$$

The strain components in a concrete specimen, subjected to a constant sustained compressive stress σ_c applied at time t_0 and held at a constant temperature, are depicted in Figure 97. Once the concrete curing concludes (t = 0 in Figure 97), shrinkage strains begin to emerge, gradually increasing at a diminishing rate. When the compressive stress is applied, the strain diagram exhibits an immediate jump (representing instantaneous or elastic strain), followed by a slower, progressive increase in strain due to creep. Accurately predicting the time-dependent behaviour of a concrete member requires precise calculations of these strain components at key locations. This prediction depends not only on the stress history of the member but also on reliable data related to its material properties. The stress history itself is influenced by both applied loads and the boundary conditions imposed on the member. Traditionally, studies examining concrete deformation differentiate between creep and relaxation problems. In creep problems, the focus lies on calculating the gradual increase in strain under sustained stress. On the other hand, relaxation problems involve holding the total strain constant over time and determining how the stress evolves gradually. The distinction between creep and relaxation problems stems primarily from their boundary conditions.

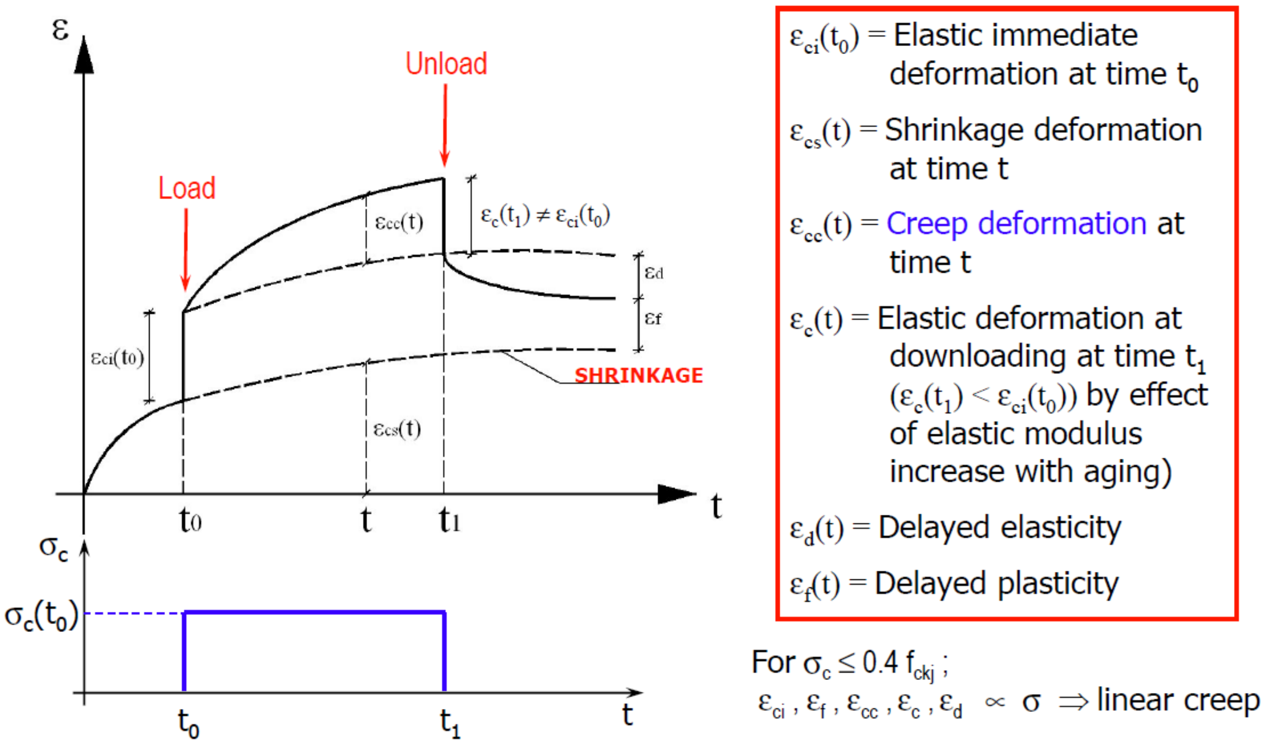

Figure 97 Concrete strain components under sustained load.

Establishing the significance of each concrete strain component from the very beginning is crucial (Figure 98).

The creep deformation at time t from a stress applied at t_0 is:

$$\varepsilon_{cc}(t) = \frac{\sigma_c(t_0)}{E_{ci}}\varphi(t, t_0) \tag{29}$$

where $\varphi(t, t_0)$ is the creep coefficient (adimensional) and E_{ci} is the Young Modulus at reference time (28 days).

The deformation due to the stress applied (creep + elastic deformation) is:

$$\varepsilon_{c\sigma}(t, t_0) = \sigma_c(t_0)\left[\frac{1}{E_c(t_0)} + \frac{\varphi(t,t_0)}{E_{ci}}\right]\varphi(t, t_0) = \sigma_c(t_0)\, J(t, t_0) \tag{30}$$

where $E_c(t_0)$ is the Young Modulus at time t_0 and $J(t, t_0)$ is the creep function, which represents the total deformation at time t by the effect of a unit stress.

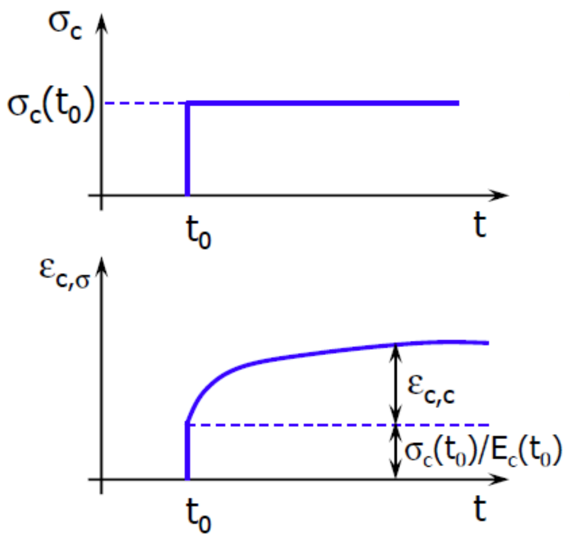

Figure 98 Diagrams of stress and strain components over the time for creep.

By considering a generic stress history (Figure 99), the application also of shrinkage (stress independent strain) and by applying the superposition principle, it's possible to get the total strain:

$$\varepsilon_c(t, t_0) = \varepsilon_{cn}(t) + \sigma_c(t_0)\, J(t, t_0) + \int_{t_0}^{t} J(t, t_0)\, \frac{\partial\, \sigma_c(\tau)}{\partial \tau}\partial\tau \tag{31}$$

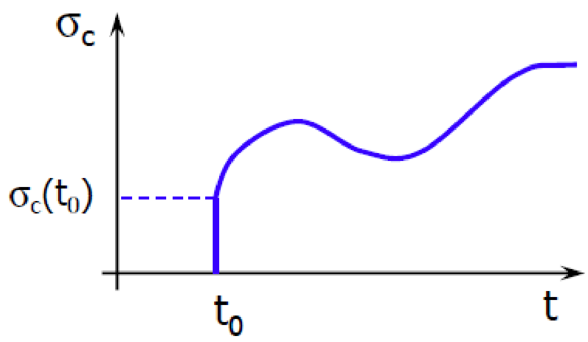

Figure 99 Generic stress field applied.

On the other hand, relaxation is the opposite phenomena respect creep. In creep is applied a constant stress and is observed an increase of deformation in time; while in relaxation is applied a constant deformation and is observed a decrease of stress in time in order to maintain this deformation constant.

In the latter case of relaxation by applying a constant deformation (Figure 100), the stress can be got as:

$$\sigma_c(\mathrm{t},\mathrm{t}_0) = \varepsilon_{c\sigma}(\mathrm{t}_0)R(t,\mathrm{t}_0) = \frac{\sigma_c(\mathrm{t}_0)}{E_c(\mathrm{t}_0)}R(t,\mathrm{t}_0) \tag{32}$$

where $R(t,\mathrm{t}_0)$ is the relaxation function, which represent the stress at time t caused by a unit-imposed deformation applied at time t_0.

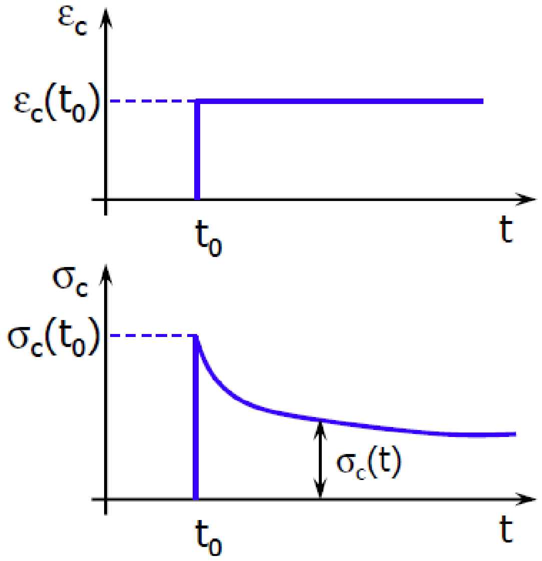

Figure 100 Diagrams of stress and strain components over the time for relaxation.

By considering a generic strain history (Figure 101), the application also of shrinkage (stress independent strain) and by applying the superposition principle, it's possible to get the total stress:

$$\sigma(\mathrm{t},\mathrm{t}_0) = [\varepsilon_c(\mathrm{t}_0) - \varepsilon_{cn}(\mathrm{t}_0)]R(t,\mathrm{t}_0) + \int_{\mathrm{t}_0}^{t} R(t,\tau)\frac{\partial(\varepsilon_c(\tau) - \varepsilon_{cn}(\tau))}{\partial\tau}\partial\tau \tag{33}$$

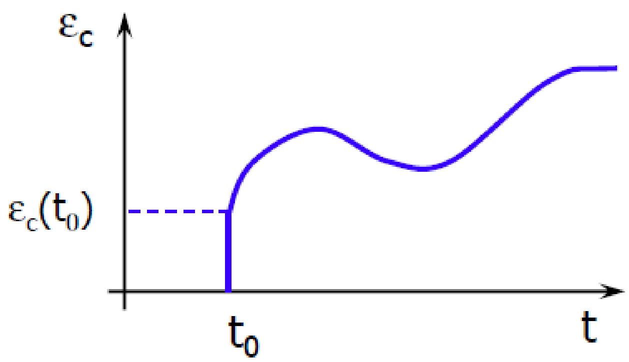

Figure 101 Generic strain field applied.

Third principle of linear creep – Principle or reintroduction of delayed restraints

A visco-elastic body, homogeneous and with n rigid restraints is subjected to $X_i(t_0)$ reactions (i=1÷n) by effect of permanent load (*g*) applied at time t_0 (Figure 102) [29].

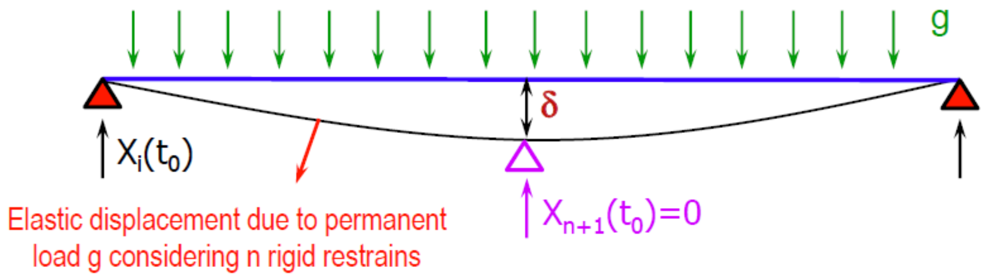

Figure 102 Visco-elastic body, homogeneous and with n rigid restraints subjected to permanent load applied at time t_0.

Immediately after the application of the load (t=t_0+), a further restraint is introduced, in which obviously at time t_0 the reaction is $X_{n+1}(t_0)$ = 0.

Let's analyse the evolution of all the reactions by effect of creep (Figure 103).

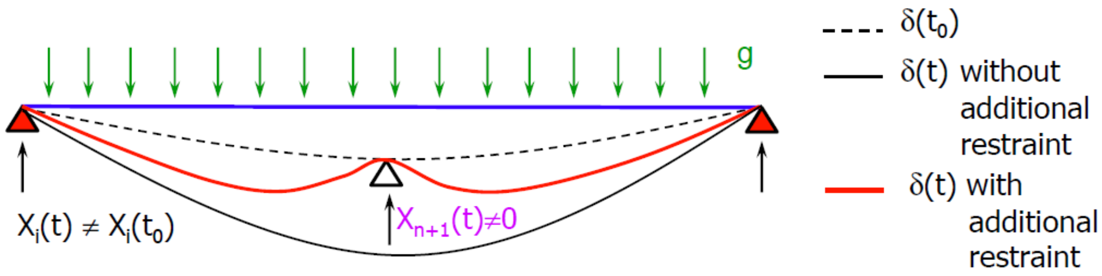

Figure 103 Evolution of deformations in the structure.

In order to evaluate the reactions, assume to get the configuration at $t = t_0$ in two (elastic) steps:

1. Suppose to introduce the n+1 restraint before the application of loads; consequently, a reaction $X_{n+1}(t_0)$ (different from zero) will arise and all other reactions will have variations $\Delta X_i(t_0)$;
2. To come back to the actual condition at time t_0, is imposed a settlement δ in the n+1 restraint, equal to the displacement evaluated at the n+1 restraint in the structural configuration with n restraints at time t_0 (Figure 104).
   Then at t_0 the state is:
   - In restraint n+1:
   $$X_{n+1}(t_0) - X_{n+1}(t_0) = 0 \tag{34}$$
   - In restraint i:
   $$X_i(t_0) + \Delta X_i(t_0) - \Delta X_i(t_0) = X_i(t_0) \tag{35}$$

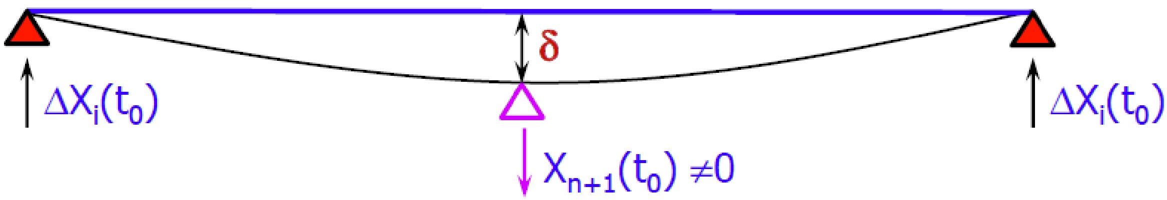

Figure 104 Application of a settlement δ in the n+1 restraint in order to come back to the initial configuration.

The two-steps procedure allows to evaluate the evolution of reactions in time, considering the different nature of the two contributions. In particular:

- Reactions evaluated in step 1 are given by applied forces: these reactions will be constant in time according to the "Corollary of the theorem of isomorphism";
- Reactions evaluated in step 2 are given by an imposed deformation (settlement δ): these reactions will be variable in time with relaxation law according to the "Theorem of isomorphism".

Consequently:

$$X_{n+1}(t) = X_{n+1}(t_0) - X_{n+1}(t_0)\frac{R(t,\mathrm{t}_0)}{E_c} = X_{n+1}(t_0)\left[1 - \frac{R(t,\mathrm{t}_0)}{E_c}\right] \tag{36}$$

$$X_i(t) = X_i(t_0) + \Delta X_i(t_0) - \Delta X_i(t_0)\frac{R(t,\mathrm{t}_0)}{E_c} = X_i(t_0) + \Delta X_i(t_0)\left[1 - \frac{R(t,\mathrm{t}_0)}{E_c}\right] \tag{37}$$

It's evident that if there is a variation of reaction *R* in time there will be also a variation of applied internal action *S*. This can be evaluated as follows:

$$S(t) = S_i + \Delta\mathrm{S}\left[1 - \frac{R(t,\mathrm{t}_0)}{E_c}\right] = S_f\left[1 - \frac{R(t,\mathrm{t}_0)}{E_c}\right] + S_i\,\frac{R(t,\mathrm{t}_0)}{E_c} \tag{38}$$

Where S_i is the internal action evaluated in the initial static scheme (without additional restraint), S_f is the internal action evaluated in the final static scheme (with the addition of the intermediate support) and $\Delta\mathrm{S} = S_f - S_i$

In particular, the "Third Principle of Linear Creep" can be applied in the case of CSTCBs. These beams, as highlighted before in Table 13, are characterised by the change of the static scheme passing from isostatic to continuous one. It's possible to make a computation to evaluate the variation of the bending moment during the different phases by considering a simple static scheme where the self-weight g_1 is subjected to the delayed restraint (Figure 105).

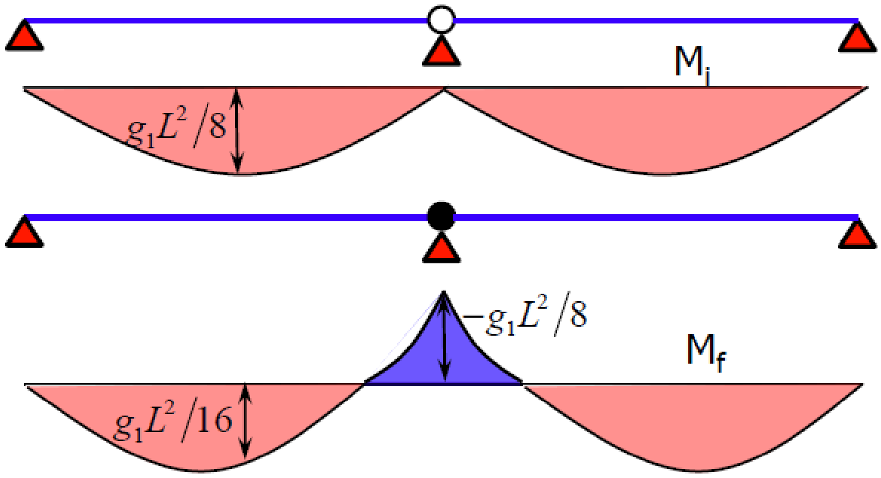

Figure 105 (top) Initial static scheme with open hinge; (bottom) final static scheme when the hinge is closed.

Initially the trussed beam is in phase I (composed only by bottom chord and steel truss) and is laid in situ between two pillars, getting a simply supported configuration. So, it's possible to say that the central hinge is opened. Then concrete is casted and this configuration is valid up to the hardening.

After the hardening of concrete, the central hinge at the bearing is closed, the static scheme becomes a continuous one and the bending moment diagram evolves consequently.

It's possible to make an evaluation on the values of bending moment in different sections of the beam and at different time, remembering that $\frac{R(\infty,\mathrm{t}_0)}{E_c}$ = 0,3 from the diagram of relaxation function by considering t_0 =28 days (Figure 106).

At $x = \frac{L}{2}$:

$$M_{g1}(\mathrm{t}_0) = \frac{g_1L^2}{8} = 0{,}125 g_1 L^2 \tag{39}$$

$$M_{g1}(\infty) = \frac{g_1L^2}{16}\left[1 - \frac{R(\infty,\mathrm{t}_0)}{E_c}\right] + \frac{g_1L^2}{8}\frac{R(\infty,\mathrm{t}_0)}{E_c} = 0{,}08125 g_1L^2 \tag{40}$$

At $x = L$:

$$M_{g1}(\mathrm{t}_0) = 0 \tag{41}$$

$$M_{g1}(\infty) = -\frac{g_1L^2}{8}\left[1 - \frac{R(\infty,\mathrm{t}_0)}{E_c}\right] + 0 = -0{,}0875 g_1L^2 \tag{42}$$

It's evident the phenomenon of stress redistribution due to creep over the time: in the hogging zone the stress increase in time in absolute value, while in the sagging zone it decreases. Therefore, the hogging zone will be more stressed at final time while sagging zone at initial time.

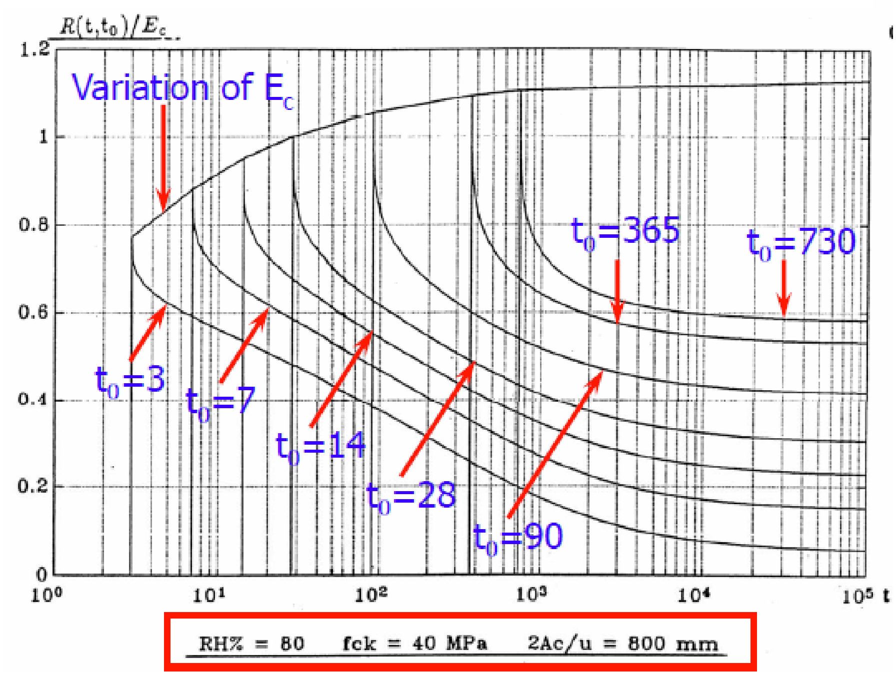

Figure 106 Diagram of the relaxation function against the time.

A final consideration is that has been possible to apply the "Isomorphism Theorems" because the structure has been considered homogeneous, assimilating the CSTCB as a RC beam where the steel truss is represented by the stirrups.

4. CASE STUDY: USE OF PRECAST BEAMS (CSTCBs) IN A BUILDING IN CIRIE’

4.1GENERAL DESCRIPTION OF THE BUILDING AND OF THE CONSTRUCTION SITE

In order to support the theoretical and technical analysis of previous chapters in the practical use of CSTCB, this chapter deals with a field application of CSTCB in an actual construction project. The present paragraph refers to the construction of a building situated in Ciriè (TO), Via Matteo Pescatore 18/22, where CSTCBs were taken as structural element for the floors system in some particular cases. The section starts with an overview of the building and its structure before examining in depth CSTCBs as employed in the project.

The construction taken as practical example for this thesis has been and is currently being designed by “ME Engineering S.R.L.”, which is a design company, made of about 45 professionals with multidisciplinary competencies, from architects to engineers. I’ve made my internship in this society in the previous months and during the drafting of this thesis I’m currently working here, participating actively to the design of the case study described.

The object of construction is a building articulated in 5 floors above the ground and 2 below the ground to form a car park. The building fulfils both the commercial and residential function (Figure 107): at the ground floor is present a supermarket, above the commercial area is present a residential building composed by 5 floors above the ground. The car parks of both the portions are situated in the two underground floors (called “floor -1” and “floor -2”).

Figure 107 Render of the structure located in Ciriè with specific focus to the commercial part (ground floor) and the residential one (upper floor).

The first stages of construction involved the demolition of an old cinema that was present in the site. Then started the real construction process.

The construction site is pretty particular, because is in a residential area and extends its boundaries up to the adjacent properties. Due to this aspect the excavation has been quite difficult: there was no space to make an ordinary excavation with angle of about 45° in the edges; so was necessary to drill micropiles in all the perimeter of the site (Figure 109 and Figure 108). These are approximately 9 - 10 m in length and has been drilled before digging out the soil and reaching an elevation of about -7,5 m from the ground floor in order to create the two underground floors for the car park of the commercial and residential areas. Ahead of the temporary berm of micropiles a 30 cm thick reinforced concrete wall has been designed in order to counteract the earth's thrust.

Figure 109 Picture from the construction site showing the micropiles used to stabilise and make safe the excavation process.

Figure 108 Picture from the construction site showing the drilling machine in action.

A slab foundation 50 cm thick is planned in order to resist to the huge vertical loads coming from the structure. The slab and the walls against the ground have to be built with additivated concrete (Penetron type) to guarantee water impermeability. The first basement floor, roof and first above ground floors are made in "predalles", RC beams and in some zones prefabricated beams. In particular, the floors of the underground and of the commercial portion need adequate fire resistance. All the remaining floors, that are those of the residential portion, are made with joists and distribution ribs and are 30 cm thick. C25/30 concrete and B450C steel are used throughout the whole building.

The structural design plans, not in scale, are provided below to give the reader more clarity about the project (Figure 110, Figure 111, Figure 112, Figure 113, Figure 114, Figure 115).

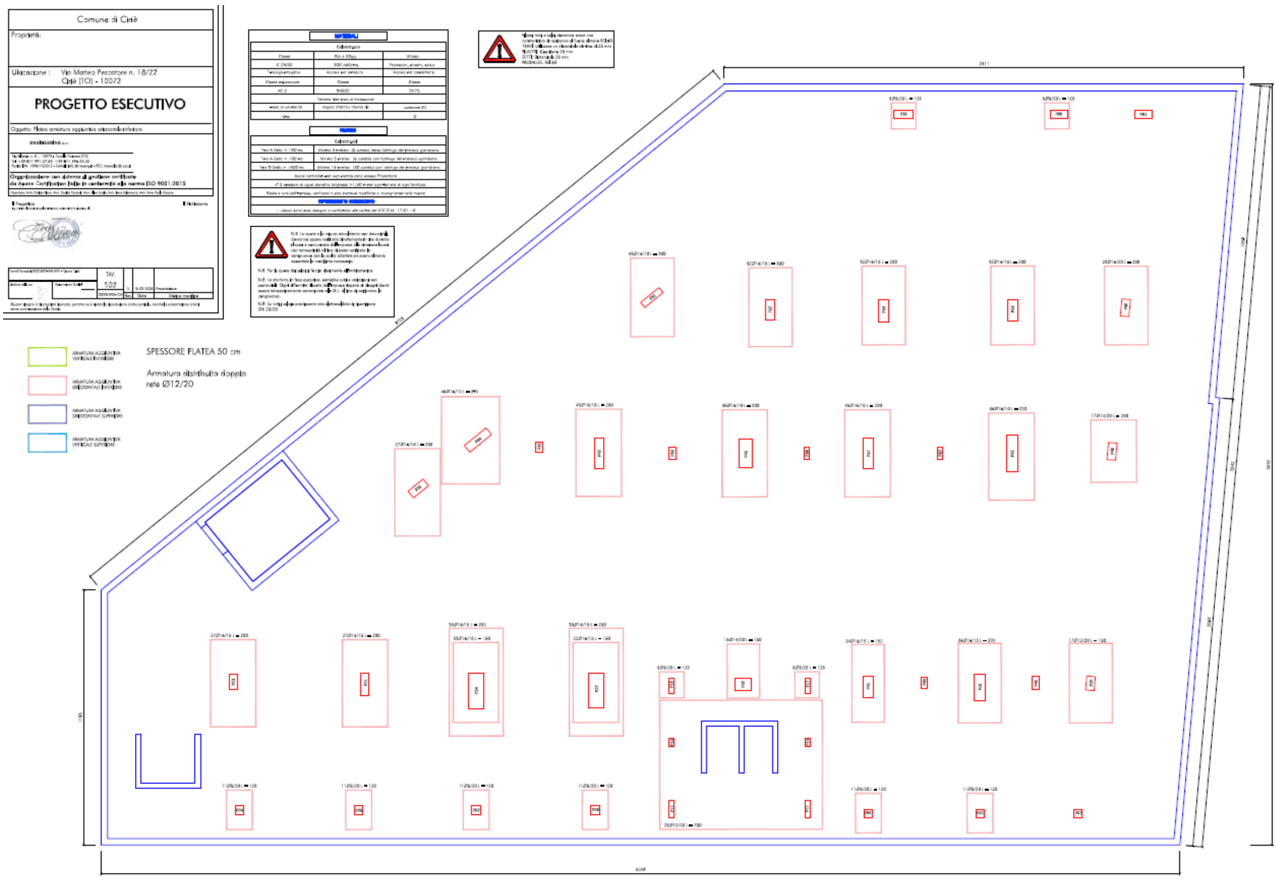

Figure 110 Structural design plan of the foundation slab.

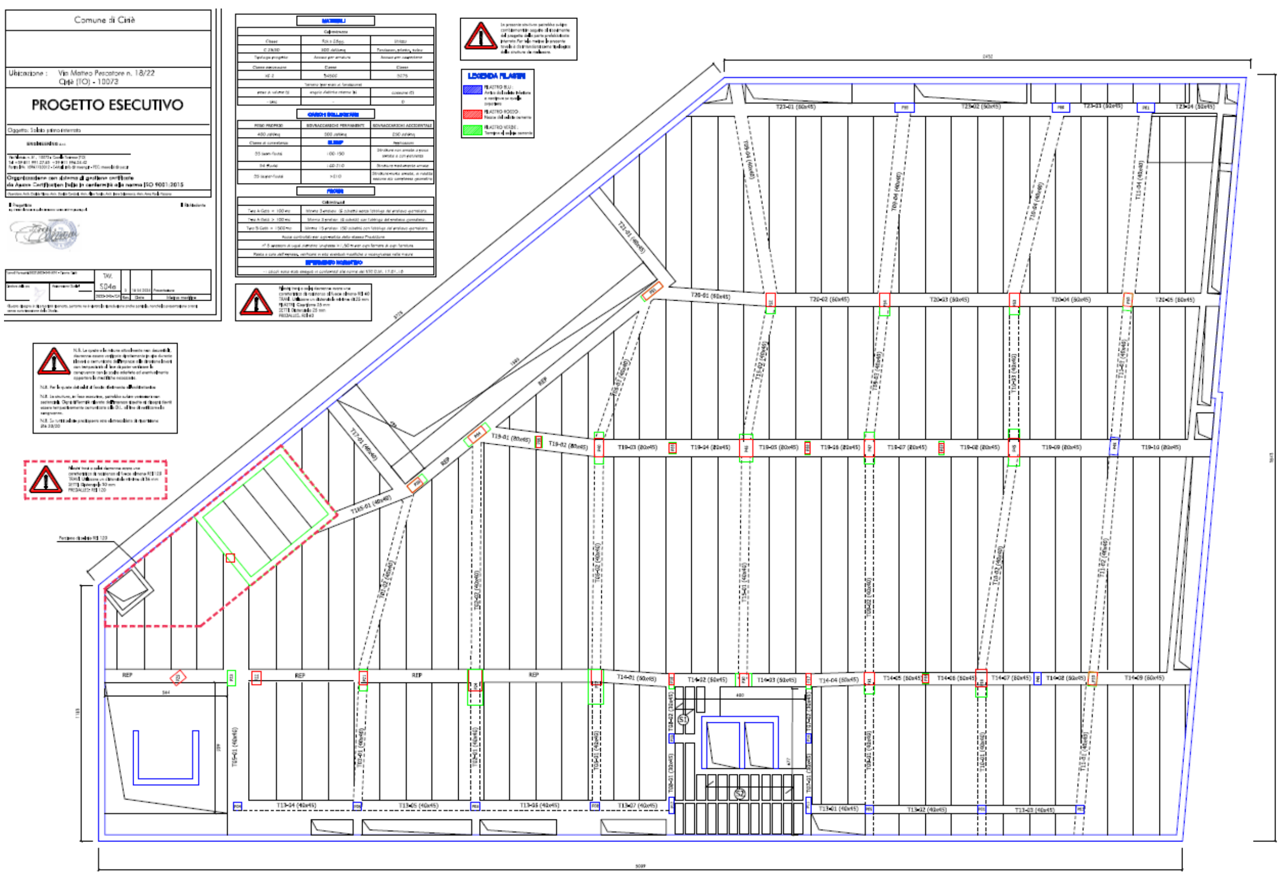

Figure 112 Structural design plan of the 1st floor below the ground.

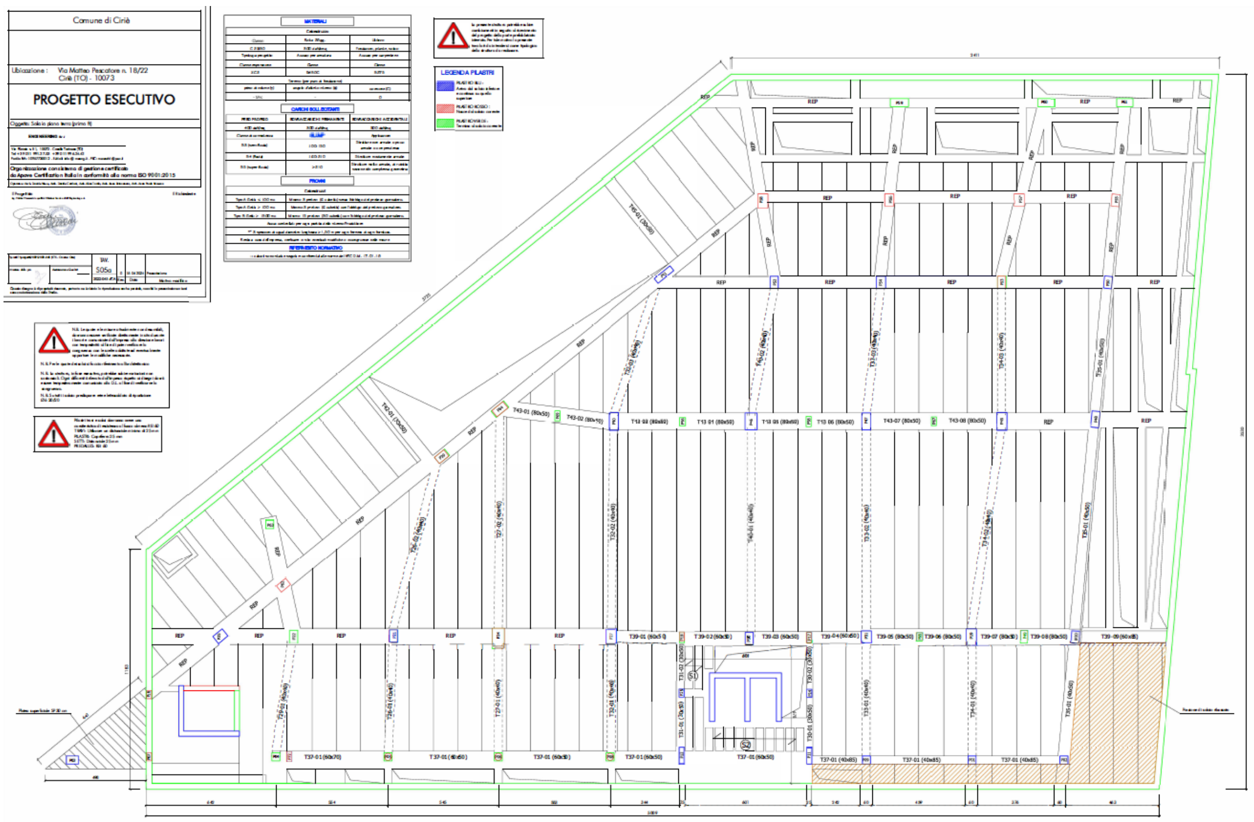

Figure 111 Structural design plan of the ground floor.

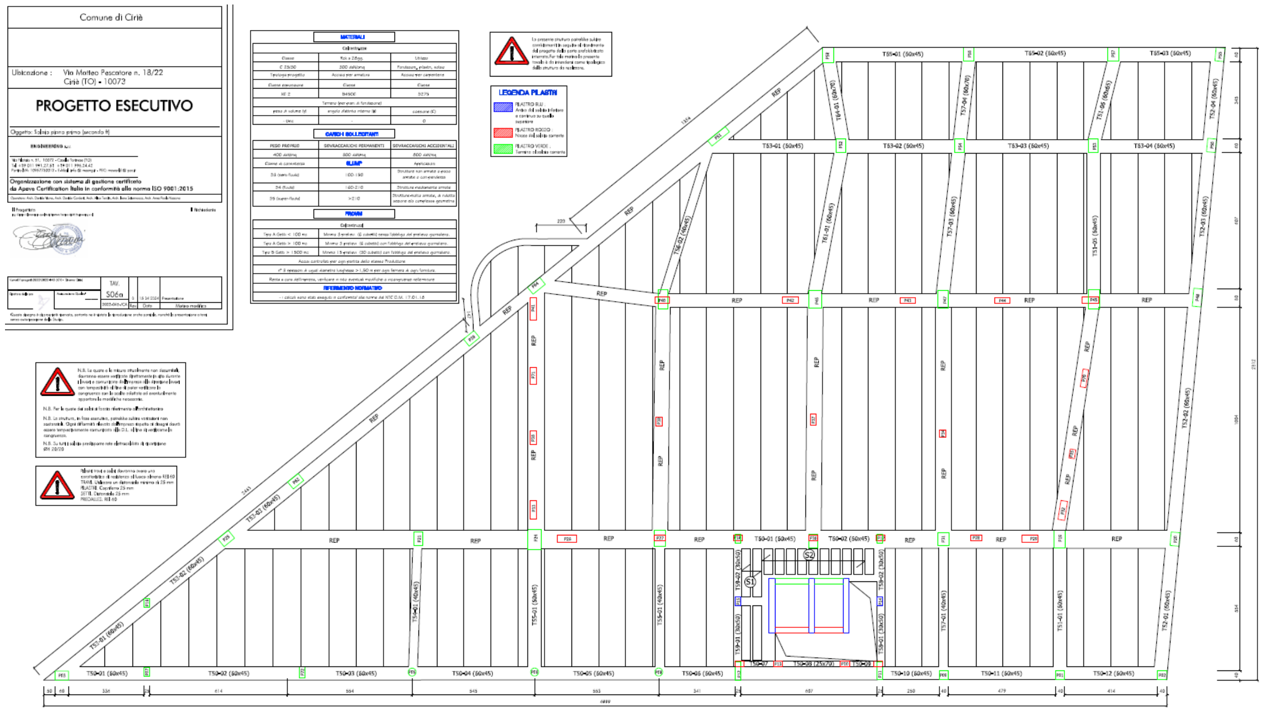

Figure 113 Structural design plan of the 1st floor above the ground.

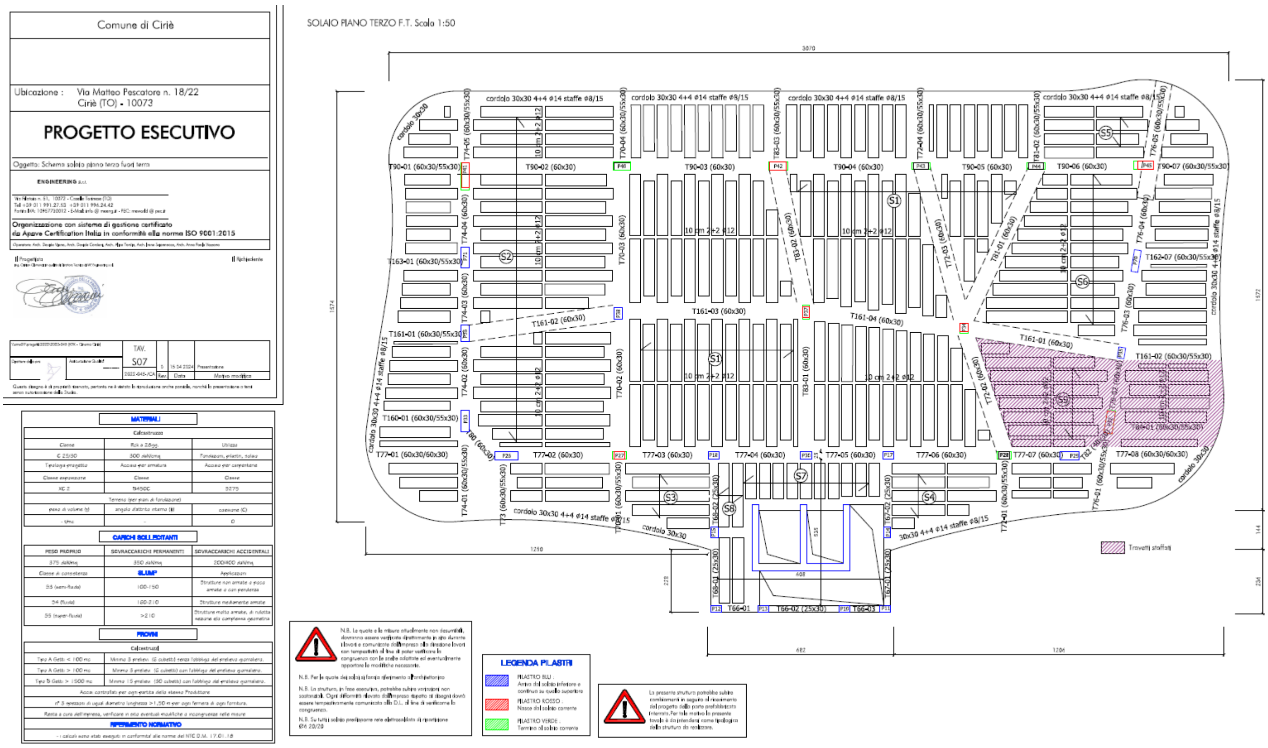

Figure 114 Structural design plan typology of the 2nd - 3rd - 4th floors above the ground.

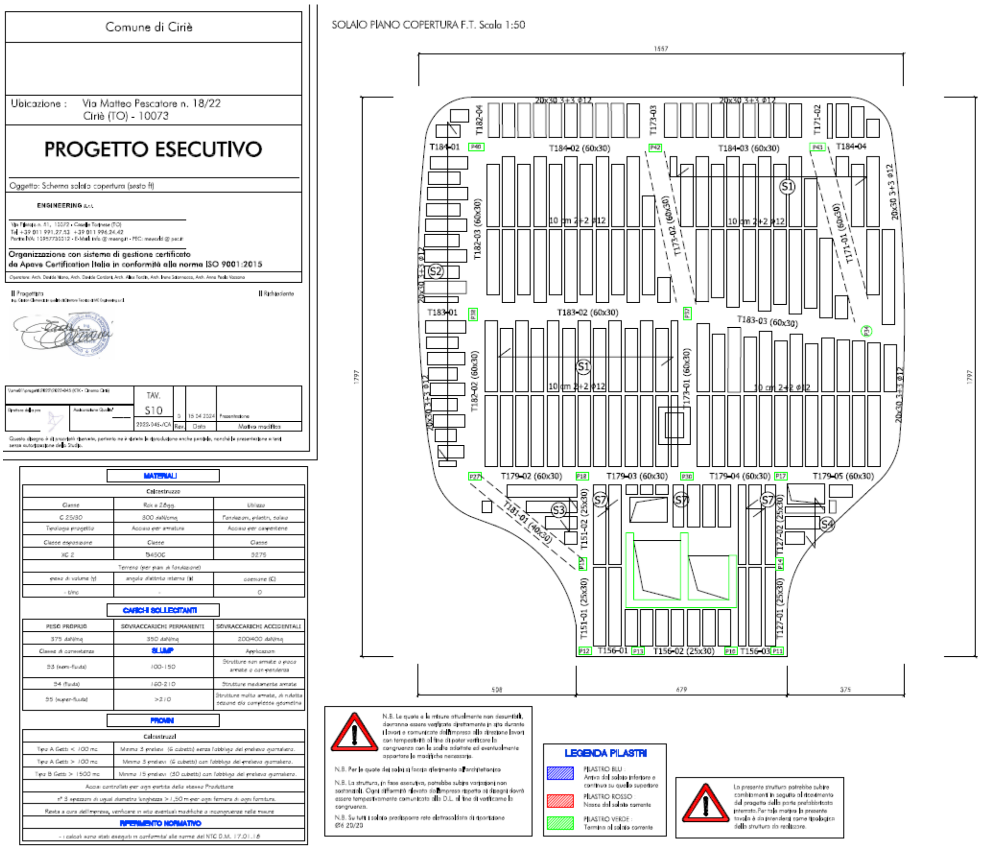

Figure 115 Structural design plan of the 5th floor (roof).

4.2PROJECT'S STRUCTURAL MODEL AND SPECIFICATIONS

The structural model of the building (Figure 116) has been developed in the software "DOLMEN".

The structure is made in reinforced concrete and has been designed for a nominal life of 50 years and a class of use II. It is a spatial frame structure and the load bearing skeleton is composed of:

- Columns: Elements with a prevailing dimension, positioned vertically between two different planes, subjected to deviated bending and biaxial shear;
- Beams: Elements with a prevailing dimension, generally positioned horizontally and belonging to only one storey and subjected to simple bending and shear;
- Foundation slab: Element with the two prevailing and comparable dimensions (X and Y), assumed on elastic soil and subjected to bending as well as to normal stress and shear coming from the pillars, constituting together with the beams, the normal framework of the building to which may be added any moments transmitted by the pillars;
- Slabs: Elements made up of with different technologies, in particular in the project are made up in "predalles" for the commercial floors and in reinforced concrete with joists for the residential floors. The slabs are defined as infinitely rigid in their plane.

The intervention is classified as a new construction in a seismic zone. The structure is schematised by excluding the contribution of the elements with negligible stiffness and resistance compared to the main ones. Therefore, the three-dimensional frame structure, the floors and the vertical partitions with high stiffness has been considered. The calculation of the structures is carried out taking into account that this is a building for civil housings as defined by the "D.M. 17 gennaio 2018 - Nuove Norme Tecniche per le Costruzioni", so the project has been developed in accordance with the prescriptions contained therein.

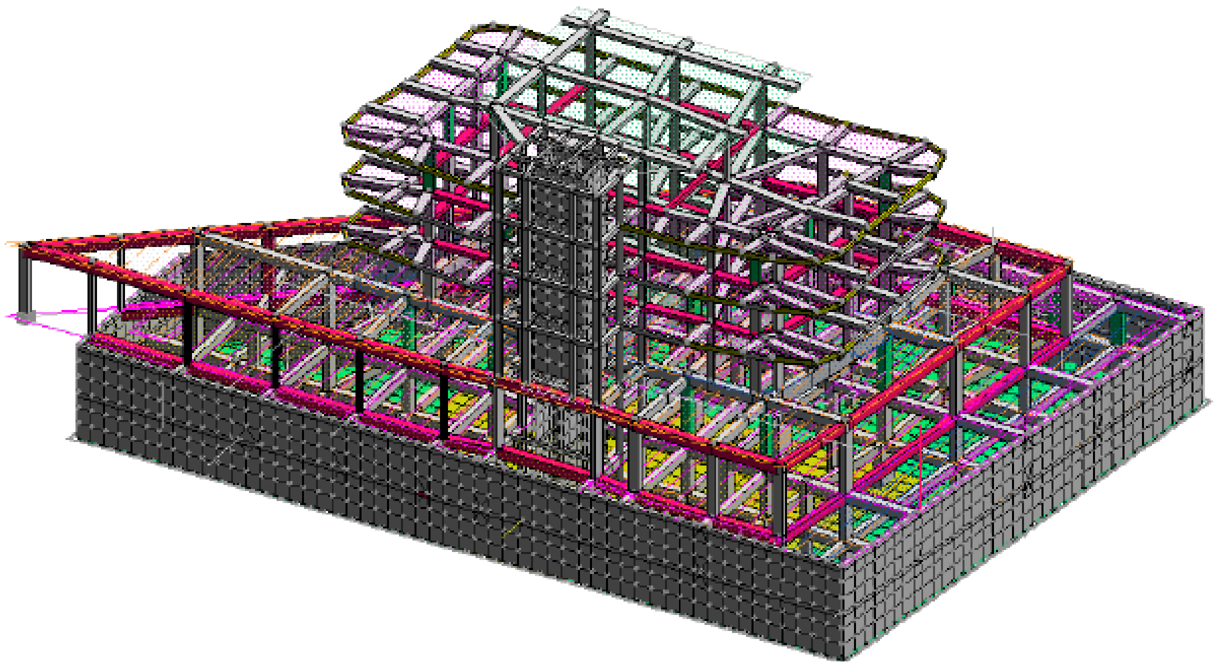

Figure 116 Assonometric view of the structural model of the structure taken from the software "DOLMEN".

The design, dimensioning and verification criteria comply with the following guidelines:

- Legge 5 novembre 1971 n. 1086 (G.U. 21 dicembre 1971 n. 321) *"Norme per la disciplina delle opere di conglomerato cementizio armato, normale e precompresso ed a struttura metallica".*
- Circ. M. n. 11951 del 14/02/1974 *"Istruzioni per le applicazione della legge n. 1086".*
- Legge 2 febbraio 194 n. 64 (G.U. 21 marzo 1974 n. 76) *"Provvedimenti per le costruzioni con particolari prescrizioni per le zone sismiche. Indicazioni progettive per le nuove costruzioni in zone sismiche a cura del Ministero per la Ricerca scientifica - Roma 1981".*
- D.M. Infrastrutture Trasporti 17 gennaio 2018 (G.U. 20 febbraio 2018 n. 42 - Suppl. Ord.) "*Norme Tecniche per le Costruzioni".*
- Circolare 21 gennaio 2019 n. 7 del Ministero delle Infrastrutture e Trasporti (G.U.n.35 del 11-2-2019 -Suppl.Ord.n.5) *"Istruzioni per l'applicazione delle 'Norme Tecniche delle Costruzioni' di cui al D.M. 17 gennaio 2018.*
- D.P.R. 6 giugno 2001 n. 380 "*Testo unico delle disposizioni legislative e regolamentari in materia edilizia".*
- UNI EN 206-1/2001 "*Calcestruzzo, prestazione produzione e conformità".*
- EUROCODES consistent with principles prescribed in NTC2018.

The design and verification of the structural elements follow the semi-probabilistic limit state method. The structure is modelled using the finite element method, applied to three-dimensional systems. The elements used are both one-dimensional (beam with possible internal disconnections) and two-dimensional (triangular and quadrangular plates and membranes). The constraints are considered punctual and inserted through the six constants of elastic stiffness. The sections subject to verification in the beams are shaped at constant pitch; the stresses in the centre of gravity of the element itself are known. The elementary load conditions are cumulated according to load combinations such that they are the most unfavourable for the individual verifications, thus determining the calculation actions to be used for the design. Loads acting on floors, derived from the load analysis, are distributed by the calculation programme automatically to the members (beams, columns, walls, slabs, etc.). The loads due to the infills, both on the foundation and floor beams, are schematised as linear loads acting exclusively on the members.

4.2.1 Load conditions

The actions are modelled by means of appropriate concentrated and distributed loads on nodes and members. The own weights of the structural elements inserted in the calculation models are self-determined by the programme, depending on the dimensions and specific weight of the material:

- $\gamma_{concrete} = 25\ \frac{kN}{m^3}$;
- $\gamma_{steel} = 78{,}5\ \frac{kN}{m^3}$;

The values of the applied loads in the different floors are subdivided into self-weight, permanent loads, variable loads and are reported below:

- <u>1st and 2nd floors below the ground:</u>
  Self-weight: $640\ \frac{daN}{m^2}$;
  Permanent loads: $200\frac{daN}{m^2}$;
  Variable loads: $250\ \frac{daN}{m^2}$;
- <u>Ground floor:</u>
  Self-weight: $640\ \frac{daN}{m^2}$;
  Permanent loads: $300\frac{daN}{m^2}$;
  Variable loads: $1000\ \frac{daN}{m^2}$ (requested by the supermarket);
- <u>1st floor:</u>
  Self-weight: $640\ \frac{daN}{m^2}$;
  Permanent loads: $300\ \frac{daN}{m^2}$ in the portion of the residential building, $1000\ \frac{daN}{m^2}$ in the zones of the garden;
  Variable loads: $200\ \frac{daN}{m^2}$;
- <u>Floors of the residential portion:</u>
  Self-weight: $330\frac{daN}{m^2}$;
  Permanent loads: $250\ \frac{daN}{m^2}$ in balconies and terraces and $300\ \frac{daN}{m^2}$ in the internal parts of the dwellings to take into account the incidence of partitions.
  Variable loads: $200\ \frac{daN}{m^2}$ in the internal parts of the dwellings, $400\ \frac{daN}{m^2}$ in balconies and terraces.

4.2.2 Limit States

The limit states considered are those defined at the paragraph 3.2.1 of the "D.M. *17 gennaio 2018",* shown in Table 3.2.1, and are:

- Life Preservation Ultimate Limit States SLV;
- Damage Limit States SLD.
- Serviceability Limit States SLE, defined by the combinations: rare, frequent and quasi-permanent.

<u>Life Preservation Ultimate Limit States SLV</u>

The actions on the construction have been cumulated in such a way as to determine load conditions such as to be most unfavourable for the purposes of the individual verifications, taking into account the reduced probability of simultaneous intervention of all actions with their respective most unfavourable values, as permitted by the standards in force. For the Ultimate Limit States, the combination adopted is:

$$\gamma_{G1}\, G_1 + \gamma_{G2}\, G_2 + \gamma_P\, P + \gamma_{Q1}\, Q_{k1} + \psi_{02}\, \gamma_{Q2}\, Q_{k2} + \psi_{03}\, \gamma_{Q3}\, Q_{k3} + \cdots \tag{43}$$

Where:

- G_1: Self-weight of all structural elements; self-weight of the ground, when relevant;
- G_2: Self-weight of all non-structural elements;
- P: Pretensioning and prestressing actions;
- Q: Actions on the structure or structural element with instantaneous values that can be significantly different over time;
- Q_{ki}: Characteristic value of the i-th variable action.
- γ: Partial coefficients as defined in Table 2.6.I of "D.M. 17 gennaio 2018";
- ψ_{0i}: Combination coefficients to take into account the reduced probability of concurrence of the variable actions with their respective characteristic values.

The resulting combinations are constructed from the characteristic stresses calculated for each elementary load condition: each variable load condition has been considered, with a rotation, as basic solicitation (Q_{k1} in the formula above).

Seismic action is combined with the other actions following the relationship:

$$G_1 + G_2 + P + E + \sum_i \psi_{2i}\, Q_{ki} \tag{44}$$

Where:

- E: Seismic action for the Limit State and for the class of importance under consideration;
- G_1: Self-weight of all structural elements;
- G_2: Self-weight of all non-structural elements;
- P: Pretensioning and prestressing action;
- ψ_{2i}: Combination coefficients to take account of the reduced probability of concurrence of variable actions;
- Q_{ki}: Characteristic value of the i-th variable action.

The effects of the seismic action are assessed by taking into account the masses associated with the following gravitational loads:

$$G_k + \sum_i \psi_{2i}\, Q_{ki} \tag{45}$$

The values of coefficients ψ are presented in Table 14:

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see EN 1991-1-1)			
Category A : domestic, residential areas	0,7	0,5	0,3
Category B : office areas	0,7	0,5	0,3
Category C : congregation areas	0,7	0,7	0,6
Category D : shopping areas	0,7	0,7	0,6
Category E : storage areas	1,0	0,9	0,8
Category F : traffic area, vehicle weight ≤ 30kN	0,7	0,7	0,6
Category G : traffic area, 30kN < vehicle weight ≤ 160kN	0,7	0,5	0,3
Category H : roofs	0	0	0
Snow loads on buildings (see EN 1991-1-3)*			
Finland, Iceland, Norway, Sweden	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l.	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H ≤ 1000 m a.s.l.	0,50	0,20	0
Wind loads on buildings (see EN 1991-1-4)	0,6	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0
NOTE The ψ values may be set by the National annex. * For countries not mentioned below, see relevant local conditions.			

Table 14 Recommended values of ψ factors for buildings.

Damage Limit State SLD

The seismic action is combined with the other actions by means of a relationship similar to the previous:

$$G_1 + G_2 + P + E + \sum_i \psi_{2i} \, Q_{ki} \tag{46}$$

Where:

- E: Seismic action for the Limit State and for the class of importance under consideration;
- G_1: Self-weight of all structural elements;
- G_2: Self-weight of all non-structural elements;
- P: Pretensioning and prestressing action;
- ψ_{2i}: Combination coefficients to take account of the reduced probability of concurrence of variable actions;
- Q_{ki}: Characteristic value of the i-th variable action.

The effects of the seismic action are assessed by taking into account the masses associated with the following gravitational loads:

$$G_k + \sum_i \psi_{2i} \, Q_{ki} \tag{47}$$

Serviceability Limit State SLE

For the verifications at Serviceability Limit State, depending on the case, are used the following relationships:

- Rare combination

$$F_d = \sum_{j=1}(G_{kj}) + Q_{k1} + \sum_{i=2}(\psi_{0i} Q_{ki}) + \sum_{h=1}(P_{kh}) \tag{48}$$

- Frequent combination

$$F_d = \sum_{j=1}(G_{kj}) + \psi_{11} Q_{k1} + \sum_{i=2}(\psi_{2i} Q_{ki}) + \sum_{h=1}(P_{kh}) \tag{49}$$

- Quasi-permanent combination

$$F_d = \sum_{j=1}(G_{kj}) + \psi_{21} Q_{k1} + \sum_{i=2}(\psi_{2i} Q_{ki}) + \sum_{h=1}(P_{kh}) \tag{50}$$

Where:

- G_{kj}: Characteristic value of the j-th permanent action;
- P_{kh} : Characteristic value of the h-th prestressing or prestressing action;
- Q_{k1}: Characteristic value of the basic variable action of each combination;
- Q_{ki}: Characteristic value of the i-th variable action;
- ψ_{0i}: Coefficient for defining the values of the permissible actions of short duration but still significant in regard to the possible concurrence with other variable actions;
- ψ_{1i}: Coefficient for defining the values of the permissible actions at the fractiles of order 0.95 of the distributions of the instantaneous values;
- ψ_{2i}: Coefficient for defining the quasi-permanent values of the permissible actions at the mean values of the distributions of the instantaneous values.

The values of the coefficients ψ_{0i}, ψ_{1i}, ψ_{2i} are contained in the table already shown for SLV. Similarly to the SLU case, the resulting combinations were constructed starting from the characteristic stresses calculated for each loading condition; in turn, each accidental load condition is considered as a basic stress, thus giving rise to many combined values. For each of the resulting combinations, depending on the element (beam, column, etc.), SLE verifications (stress, deformation and cracking) are carried out.

4.3 WHY CHOOSE CSTC BEAMS?

The project started with the idea of a classical design with RC structural elements, so was crated the model in DOLMEN, added the loads and then computed the stresses in all the elements. Then have been designed the beams and the columns. During the dimensioning stage of some beams was clear that in specific zones the typical RC solution was not compatible with the huge solicitations present. This was due mainly to the following aspects concerned in the design:

1. At the underground floors is necessary large clearance between the columns in order to have enough space for the car park and for some garages. This led to the design of beams with great spans (around 6 m) and heavily loaded (example 1 in Figure 118 and Figure 117).
2. Due to architectural needs, the plants change from "floor -2" to "floor -1" (underground floors) and so in some areas are present beams supporting columns (called in Italian "pilastri in falso"). These are points of irregularity of the structure and must be analysed in a deep way.
   Figure 117 for example represent a portion of the structural design plan of the "floor -1": pillars coloured in blue are those that have the continuity in the different floors, pillars in red are those that born in the current slab, while green pillars are those that end at the current slab. It's clear to see that it's present a beam supporting the columns "P25" and "P22". In the next paragraphs will be deeply analysed this particular case to focus on the structural efficiency, load carrying capacity and ability to get longer spans with reduced depths.
3. On the roof of the commercial building (called "1st floor" in the design plants) will be built the residential structure. It's acknowledged that the spans of the commercial area are greater than those of the residential one, a consequence is that the pillars of the residential house will born on the roof and will be supported "in falso" by some beams. Therefore, a skeleton made of CSTCBs is needed in order to sustain these huge punctual loads (Figure 119 and Figure 120). In this technical drawing the same convention of colours used in point 2 is maintained to describe the development of pillars.

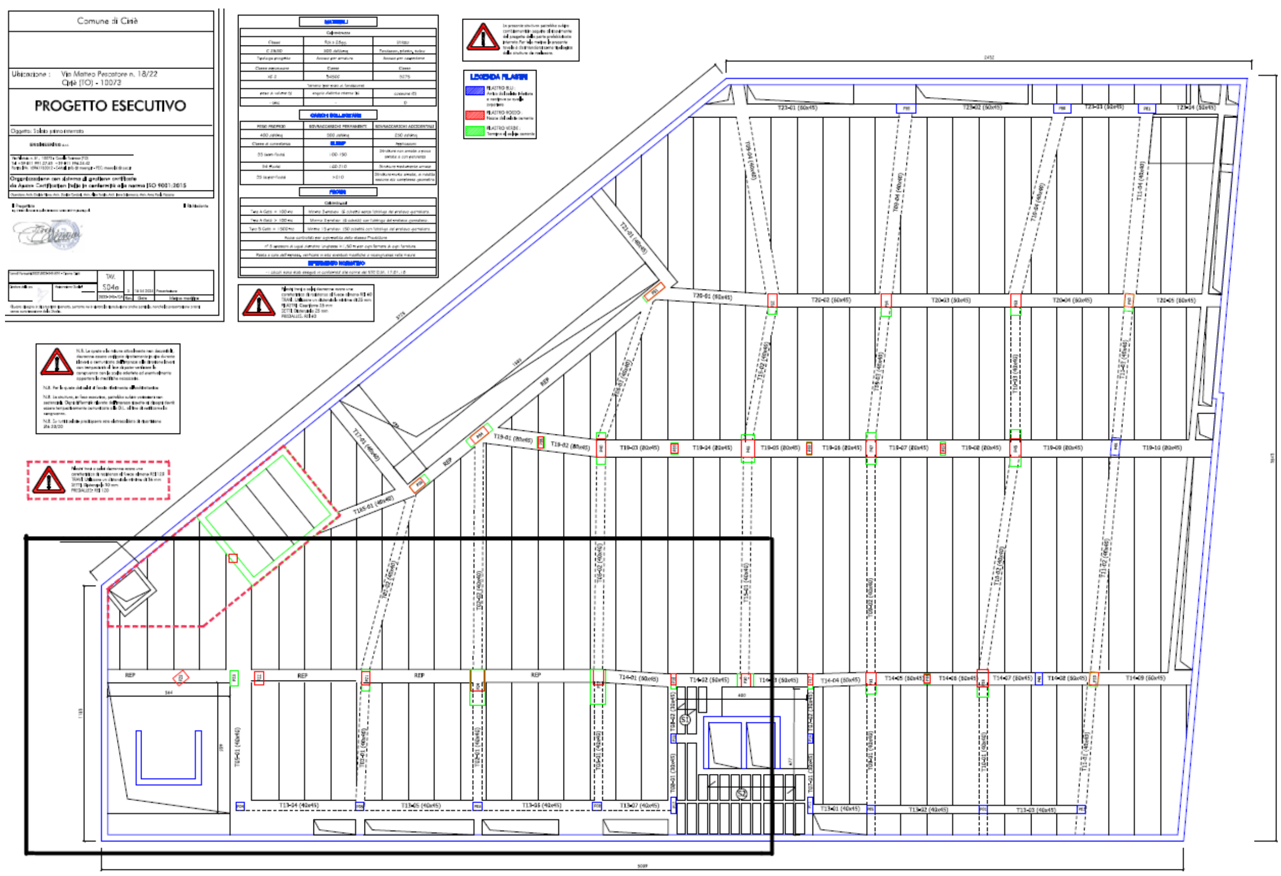

Figure 118 Structural design plan of “floor -1” to highlight the portion considered in the example in Figure 117.

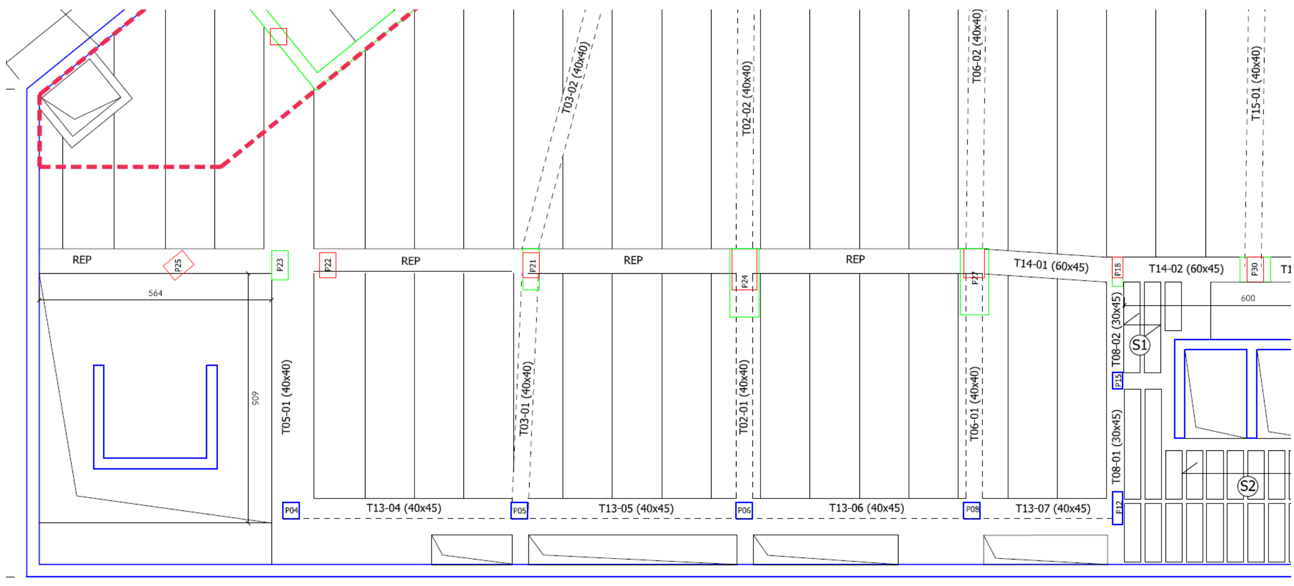

Figure 117 Portion of the structural design plan of “floor -1” where is posed particular attention to the CSTCBs (called in the technical drawing as “REP” in Italian).

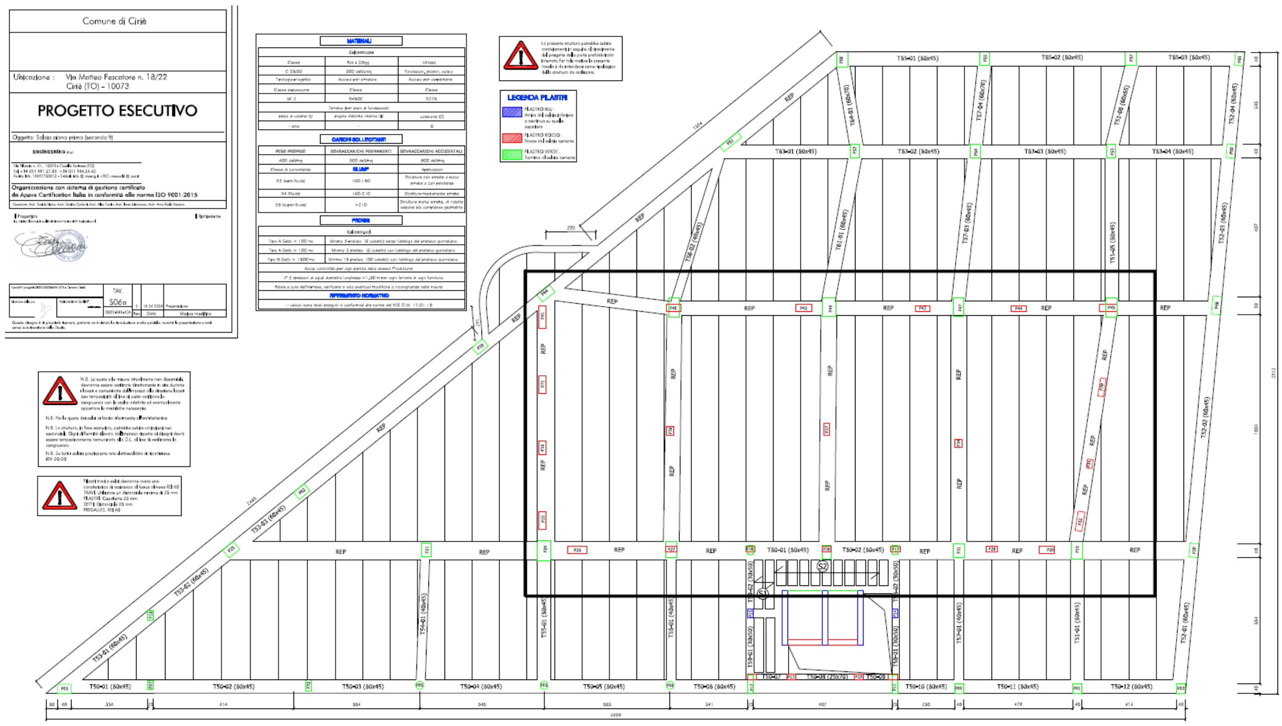

Figure 119 Structural design plan of "1st floor" to highlight the portion considered in the example in Figure 120.

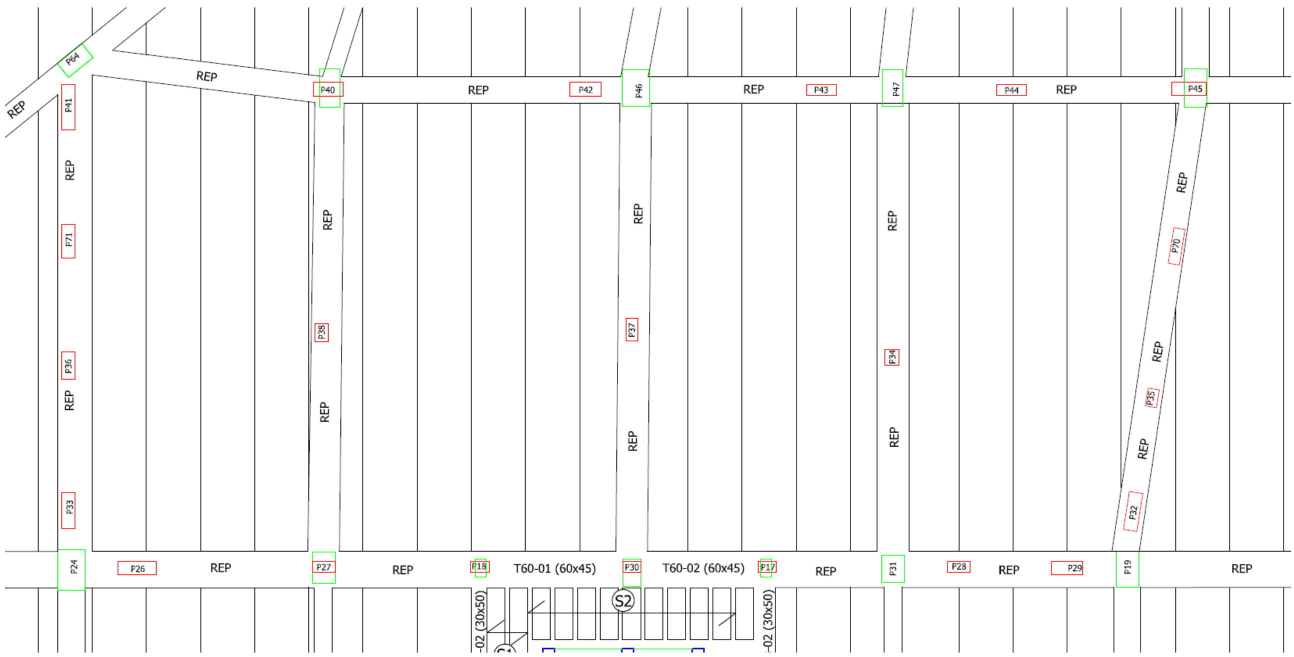

Figure 120 Portion of the structural design plan of "1st floor" where is posed particular attention to the CSTCBs (called in the technical drawing as "REP" in Italian) which sustain the pillars (in red in the technical drawing) of the residential portion of the building.

The CSTCBs used in this project are produced by "U.T. FORNACE CALANDRA s.r.l." and have a particular type of configuration made principally of (Figure 121 and Figure 122):

- A steel truss that works as a connection web welded to both the upper and lower chord. This element carries the shear loads.
- A top chord formed by at least one rebar of a specific diameter.
- A bottom chord that works as a closing element for the steel truss which is composed by numerous single steel rebars placed perpendicularly to the longitudinal axis of the beam and welded in correspondence of each step of the truss. They are not used to provide additional bending resistance but only to "close" the steel truss and promote its confinement action in order to work as a sort of stirrups.
- Additional reinforcements in the ends to provide the continuity and support the negative moment that arise passing from phase I to phase II.

This typology has been directly advised by the general contractor, that is used to build with this technology. Was decided to avoid the bottom chord in concrete ("fondello in calcestruzzo armato") mainly due to the weight savings in order to make easier the handling of the beams during the assembly on site.

Figure 121 Picture from "U.T. FORNACE CALANDRA s.r.l." production where is shown a CSTCB assembled in the factory.

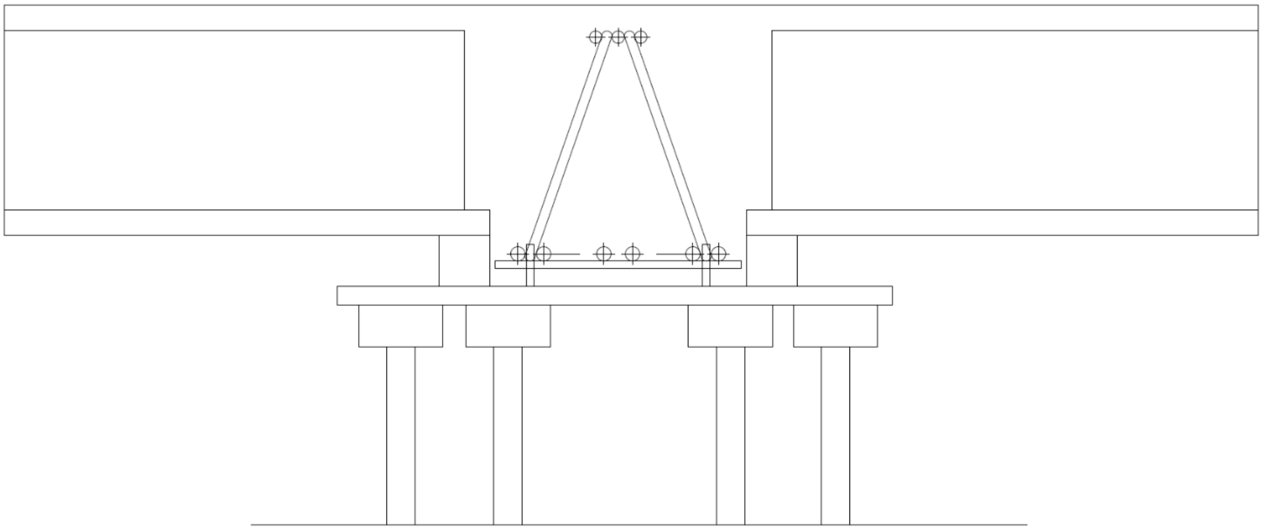

Figure 122 Technical drawing of the CSTCBs used in the project.

It’s evident that the choice of using CSTCBs in this specific project are driven mainly by their structural efficiency and the capacity to satisfy the structural needs explained above, but obviously by using these prefabricated beams it’s possible to benefit of all the advantages linked to this technology, such as:

- Fast on-site installation;
- Increase usable spaces;
- Reduction in use of formwork and scaffolding;
- Better control over quality due to arrangements and welding of reinforcements in the factory;
- Possibility to reduce the number of on-site jobs and no need for specialised workers;
- Staged construction sequence (from phase I to phase II) that simplify the process;
- Optimization of the materials, as steel is used only where needed resulting in an economical design.

4.4 COMPUTATION OF CSTC BEAMS AND COMPARISON WITH RC BEAMS

In this chapter, a detailed structural analysis is carried out to evaluate the performance of Composite Steel Trussed Concrete (CSTC) beams compared to traditional reinforced concrete (RC) beams. The goal is a direct computational comparison of CSTC technology with respect to structural efficiency.

The analysis is based on Example 1, introduced in 4.3, which represents a beam of the "floor -1" in the building located in Ciriè (TO), for a better clarity are reported again the specific technical drawings. In Figure 123 and Figure 125 pillars coloured in blue are those that have the continuity in the different floors, pillars in red are those that born in the current slab, while green pillars are those that end at the current slab.

This beam element, called "T14" in DOLMEN, has been chosen for the computations because is characterised by great spans of about 6 m and is a beam supporting columns, specifically is supporting the columns "P25" and "P22" (Figure 124 and Figure 126). It is a peculiar scenario which serves as a reference case for the dimensioning and verification of both CSTC and RC solutions under the same design loads and geometric constraints. As regards the RC beam, the design is performed with the software DOLMEN based on the ULS verifications (yet considered in 4.2). For the CSTCB to obtain an overall assessment of the behaviour construction to service the analyses are performed considering both Phase I (constructive period, the unique stage where the deck is only the steel truss to support the load) and Phase II (life of the composite beam after hardening concrete).

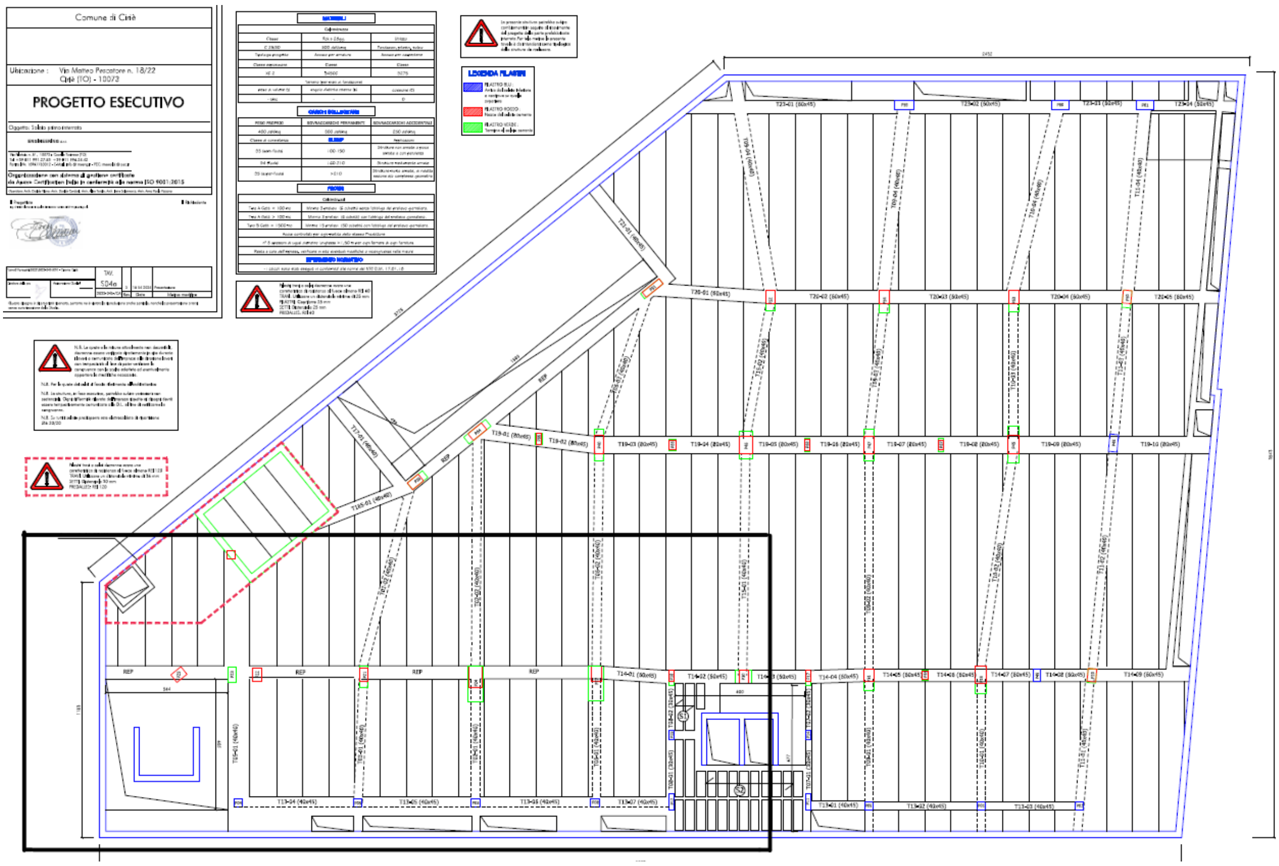

Figure 123 Structural design plan of "floor -1" to highlight the portion considered in the example in Figure 125.

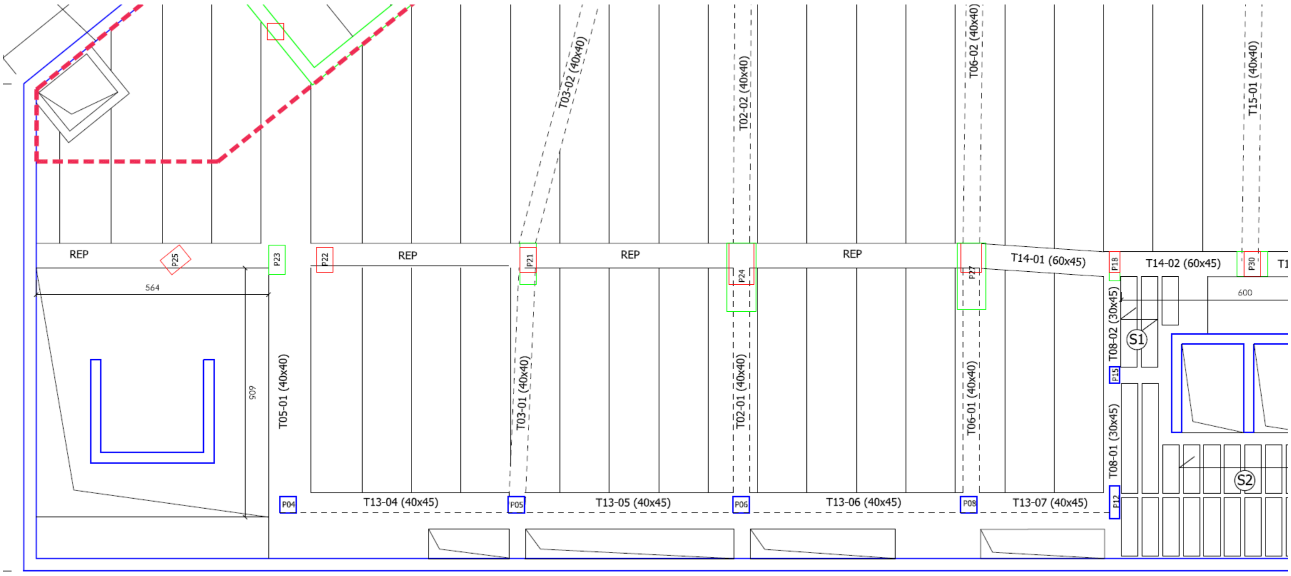

Figure 125 Portion of the structural design plan of "floor -1" where is posed particular attention to the CSTCBs (called in the technical drawing as "REP" in Italian).

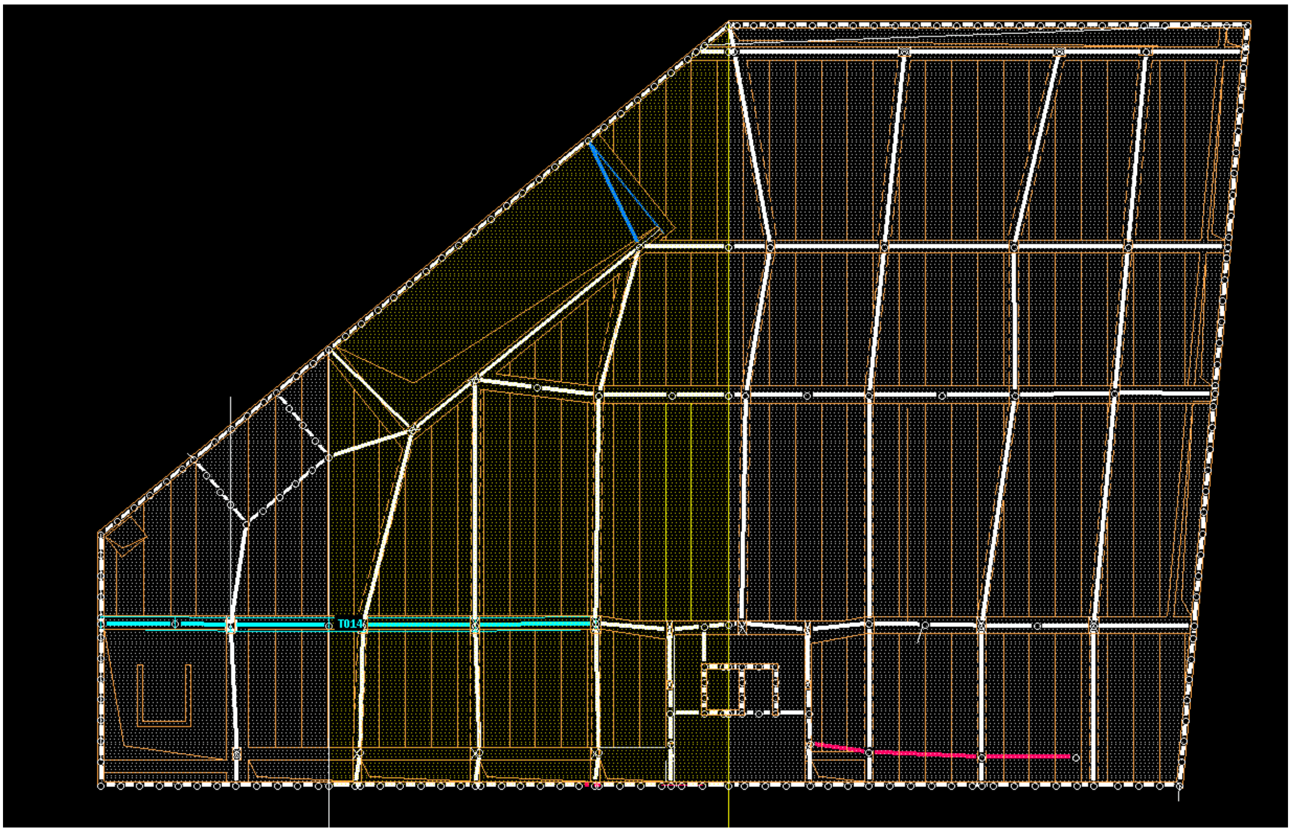

Figure 124 Screenshot from the Structural model done in DOLMEN that represents the plan of "floor -1" underlining the beam object of the computations (called "T14" in DOLMEN).

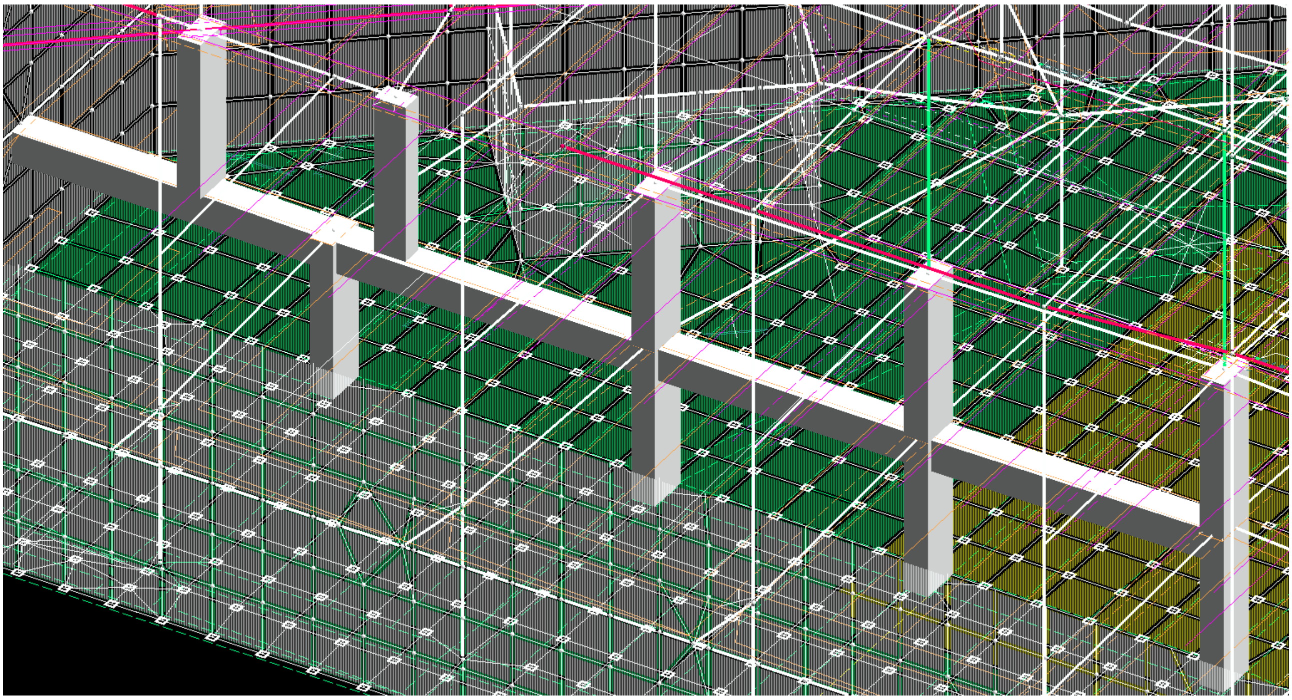

Figure 126 Screenshot from the Structural model done in DOLMEN to highlight the geometrical configuration of the beam considered for the computations.

The loads acting on "floor -1" are:

- Self-weight: 640 $\frac{daN}{m^2}$;
- Permanent loads: 200 $\frac{daN}{m^2}$;
- Variable loads: 250 $\frac{daN}{m^2}$;

These are spread on the floor, then are taken by the beams following the competence of slab on beams and finally transferred to the columns to arrive at the foundation.

Beam "T14" under analysis has a continuous configuration with 4 spans. In the first 2 spans, subjected to higher loads, the section is 50 x 75 cm. In the last two spans the section is reduced to 50 x 55 cm.

Pillars "P25" and "P22" are those that are supported by the beam and represent the most critical aspect for its design. The axial load carried by "P25" is about 100000 daN, that one carried by "P22" is about 140000 daN. These huge loads are transmitted to the beam as concentrate forces and consequently cause big peaks in the envelope of bending moments, as shown in Figure 127 Bending moment and shear diagrams for Beam T14 at ULS.

The highest values of acting bending moment are in the first span and are respectively:

- $M_{sd}^{+} = 1003 \text{ kNm}$ in correspondence of the concentrated force from pillar "P25" of the upper floor;
- $M_{sd}^{-} = 1221 \text{ kNm}$ in correspondence of the first bearing.

Below is provided the comparison between the same element designed as a RC beam or as a CSTCB.

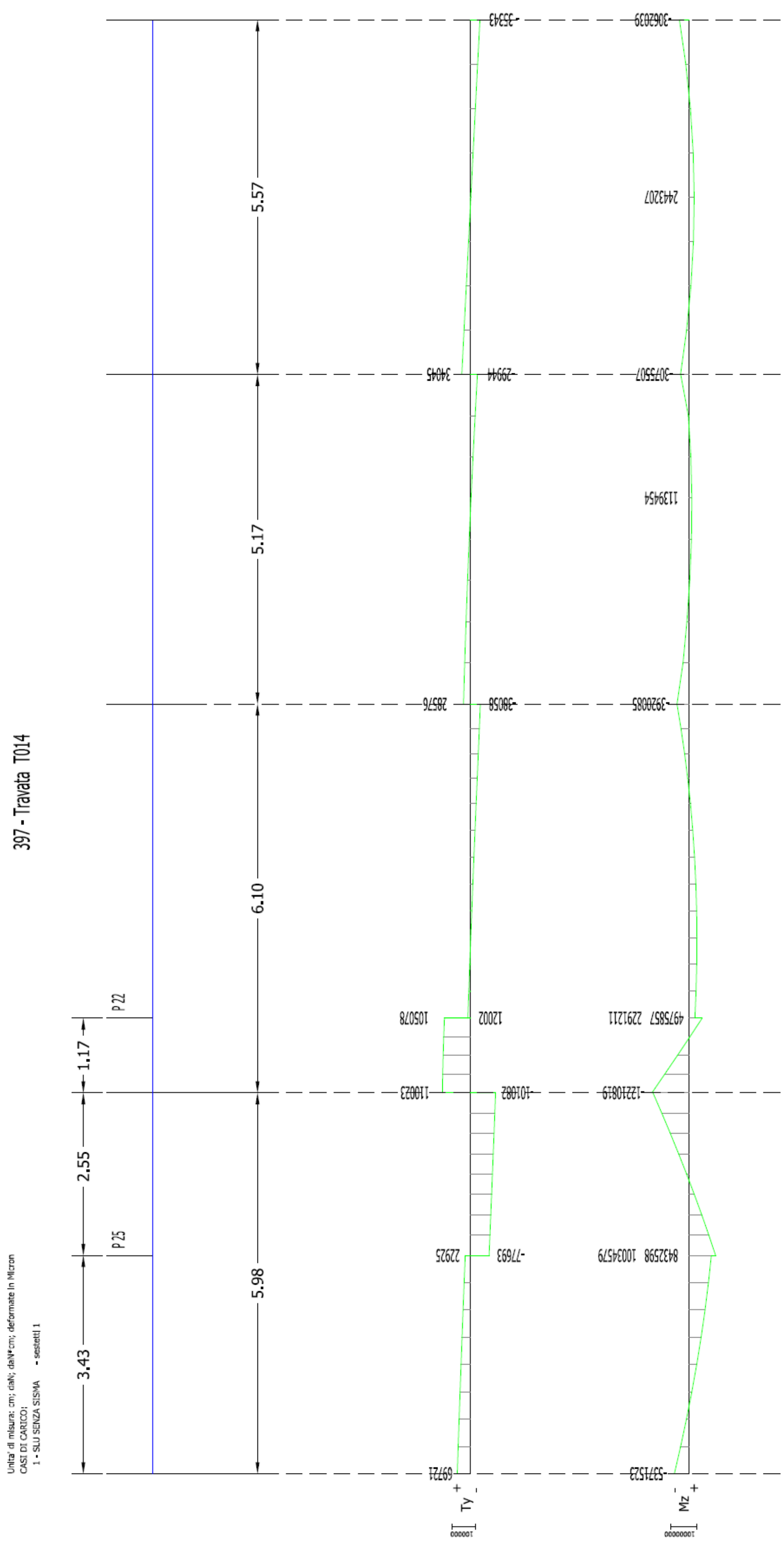

Figure 127 Bending moment and shear diagrams for Beam T14 at ULS.

4.4.1 Solution with RC beam

In this section is developed the structural design of T14 as a RC beam using the software DOLMEN. The beam is analysed at ULS under the load combinations yet described, while the software is employed to define the envelopes to the resultant internal actions, bending moments and shear force. These are compared to the resisting moment and shear capacity envelopes, determined from the mechanical properties of the material and geometry of the cross-section. The verification in Figure 128 shows that the beam satisfies the ultimate limit state (ULS) requirements in terms of both bending and shear resistance, because the envelopes of the actions (represented in blue) are contained in those of resistances (represented in green).

However, the amount and distribution of the required reinforcement result in a highly congested section. The great number of longitudinal and transverse rebars and the limited space available for rebars make it difficult in concrete pouring, in particular for concrete compaction during the construction stages. It should be added that the dense reinforcing scheme also makes the reinforcement placing operation particularly difficult and time-consuming at the site level, with the risk of delays and defects in the construct process. In particular this aspect can be highlighted in some sections:

1. In correspondence of the first span of T14 are present 12 rebars Φ22 in a section 50 x 75 cm. This led to a net distance between rebars of 1,9 cm which is of difficult feasibility on site.
2. In correspondence of the first intermediate bearing, it's present the overlapping of 12 rebars Φ22 and 5 rebars Φ20 in order to provide continuity of reinforcement and enough resistance to negative moments. This brings to an important densification of reinforcement in a small section, again not feasible on site.
   To reduce the congestion of rebars in the same plane a possible solution could be to place the longitudinal reinforcement in layers at different heights. The main drawback is that closer are the layers to the centroid of the section and less effective are in term of resisting bending moment, so at the end will be present a lot of reinforcement not used in optimal way.

For all the reasons mentioned, although the beam is compliant from a purely structural point of view, the proposed reinforcement layout is not considered feasible in practice.

An optimization of the section and of the reinforcement layout is required in order to make the construction feasible while maintaining adherence with structural safety requirements. It will be done by replacing the classic RC beams with CSTC beams.

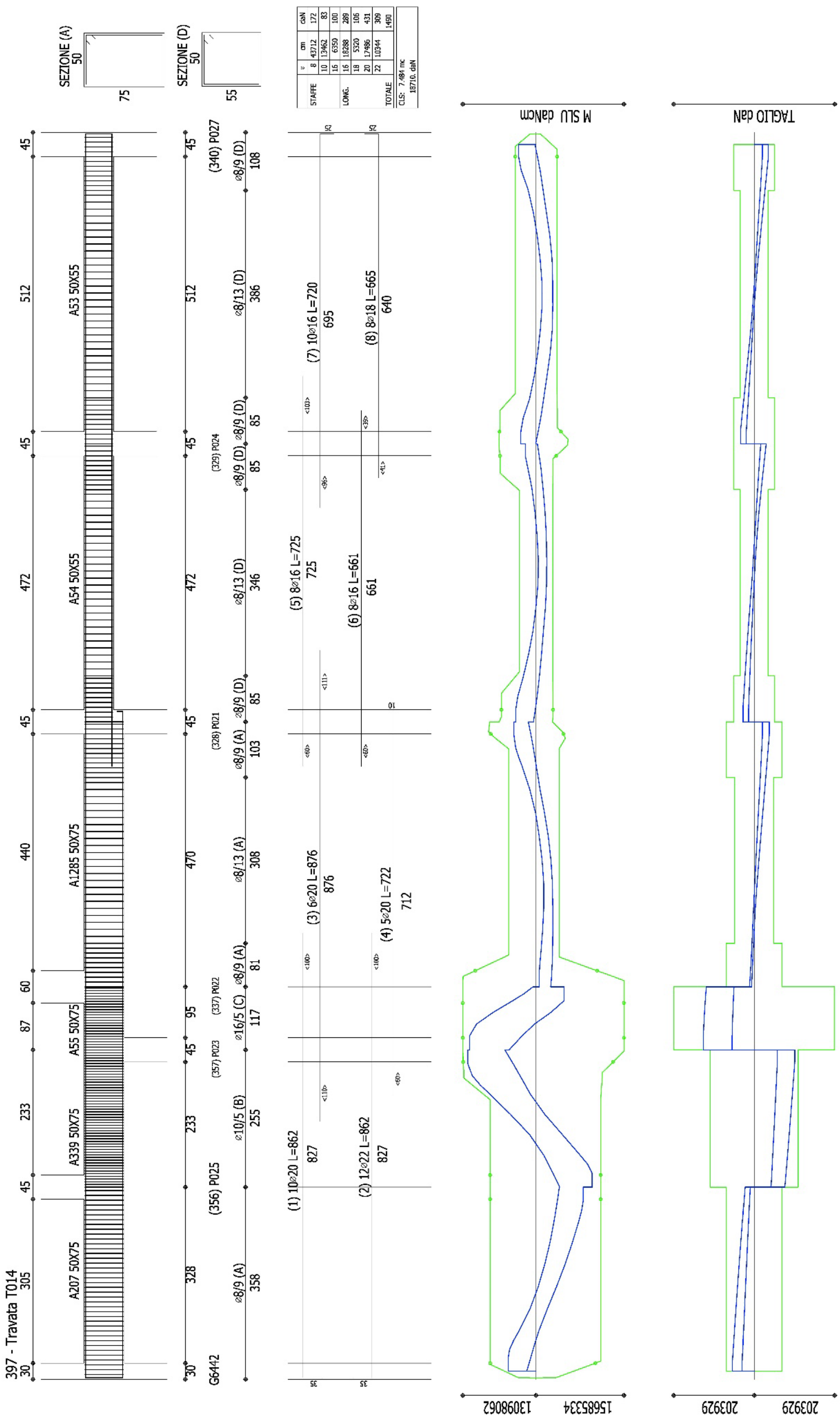

Figure 128 RC solution for beam T14: Mrd, Msd, Vrd, Vsd envelopes.

4.4.2 Solution with CSTCBs

Since the reinforcement layout coming from the design process of RC beam is not suitable in practical terms, a CSTCB is taken into account as an alternative structural solution. These are elements that can provide a really high structural efficiency and the advantage of better constructability, especially when reinforcement congestion and other placement issues are of concern. The CSTCBs studied in this work are designed and produced by the company “U.T. FORNACE CALANDRA s.r.l.” that is a corporation working for many years in the realization of these prefabricated building elements. In this specific design process, as a designer for “ME Engineering s.r.l.”, I’ve sent the envelopes of acting bending moment and shear obtained from DOLMEN to the production company. Based on these input diagrams, the company have used its proprietary software to simulate comparable loading conditions, aiming to reproduce similar envelopes of internal forces. Then with the help of the program are computed the resistances at ULS of each span in terms of bending moment and shear for both phase I (only the steel truss resists to the loads) and phase II. The resistances are obtained using the classical formulations and principles valid for RC beams, but applied to the specific case. The recreated envelopes of actions are then compared to those of resistances to assess the performance of the CSTCBs.

From a design point of view has been chosen to use CSTCB technology for the 4 spans of T14 (Figure 129). The choice in the first two spans (R101 and R102) is driven mainly by structural needs in term of resistances, while for the third and fourth spans (R103 and R104) is mostly for continuity purposes during the construction stages and so to effectively benefit of the advantages of this hybrid system on site. In particular each span of T14 has been considered separately and designed with different reinforcement. In Figure 131 is shown the section (50 x 75 cm) for beams R101 and R102 and in Figure 130 the section (50 x 55 cm) for beams R103 and R104. The complete layout is in Figure 132.

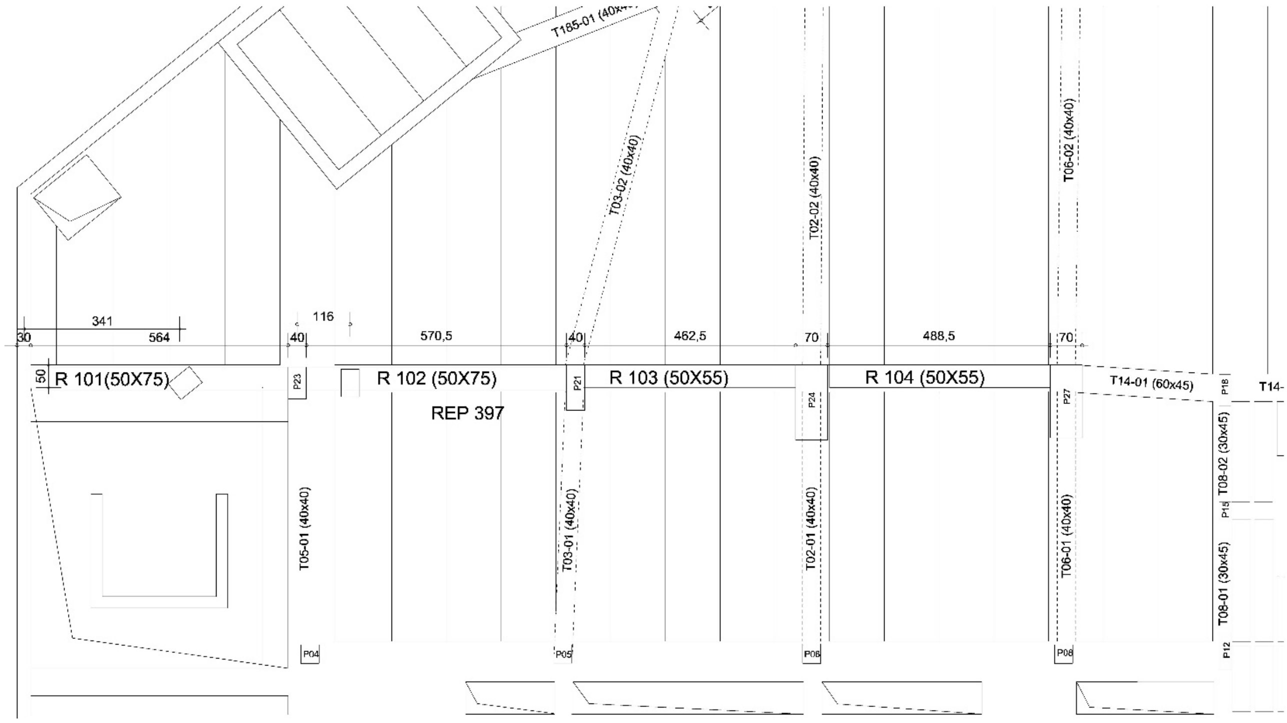

Figure 129 Structural design plan of “floor -1” to show CSTCBs R101, R102, R103, R104.

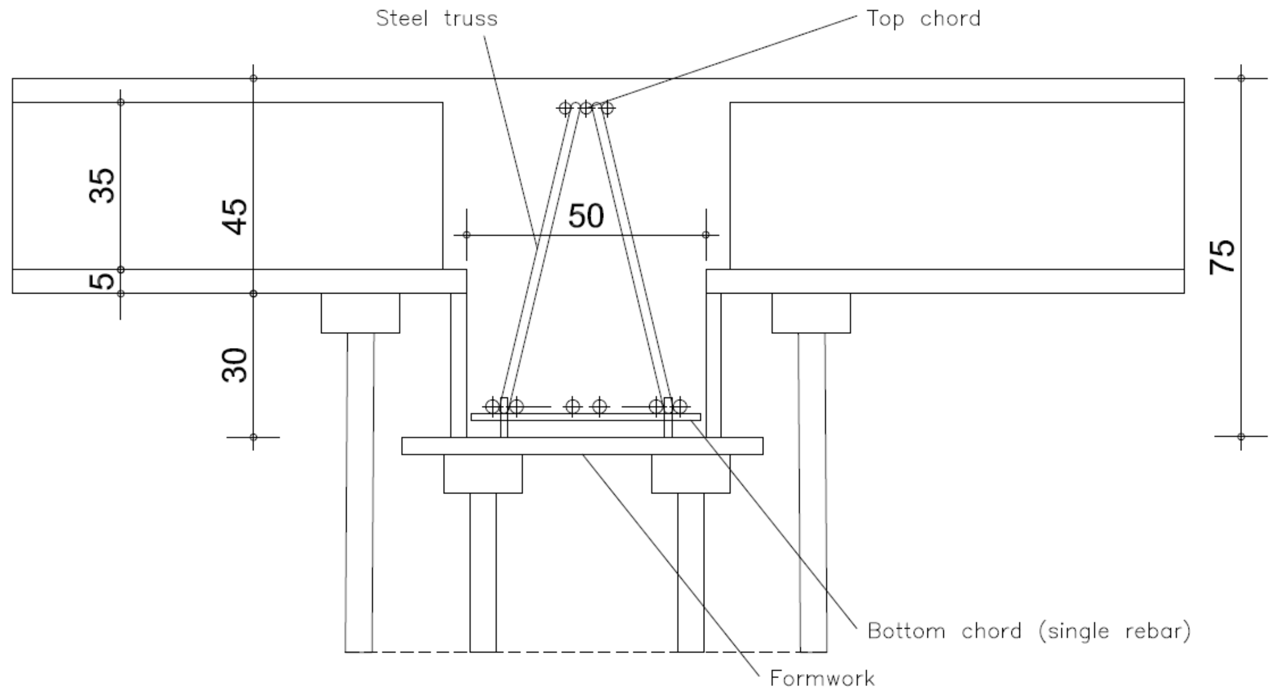

Figure 131 Technical drawing representing the cross section and layout of CSTCBs R101 and R102.

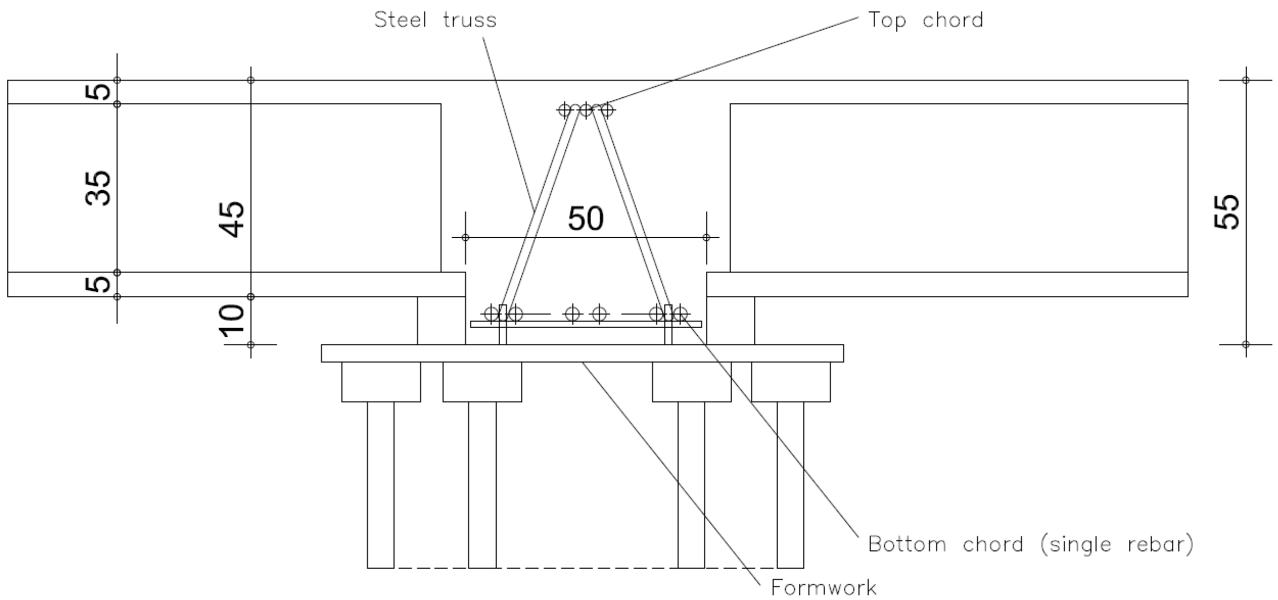

Figure 130 Technical drawing representing the cross section and layout of CSTCBs R103 and R104.

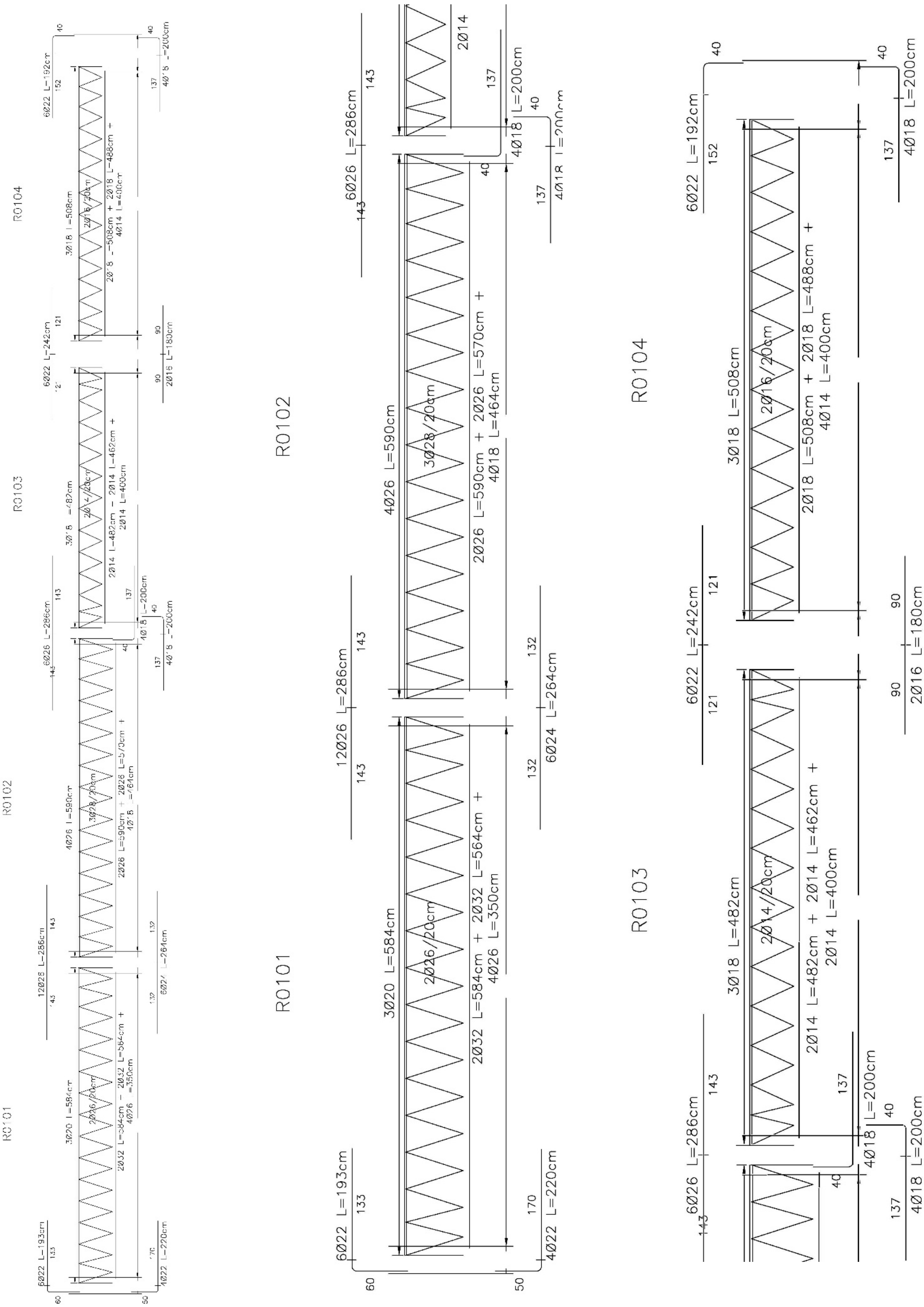

Figure 132 On the left the layout of the complete CSTCB. In centre an enlargement of the first two spans. On the right an enlargement of the third and fourth spans.

In the present report an analytical assessment of the CSTCB “T14” is carried out using hand calculations and FEM models. These estimates are based on standard procedures for design of RC beams by considering ultimate resisting moment and resisting shear force of the CSTCBs in Phase I, during the period when the steel truss predominantly carries the loads, and in Phase II, when the concrete has set and the section acts compositely as a unit. This method provides a critical comparison between the conventional RC solution and the prefabricated CSTCB considering the structural performance and the constructability.

The calculations will involve separately the flexural and the shear problems, for each span of T14 (R101, R102, R103, R104) both in phase I and in phase II.

The loads acting on “floor -1” are:

- Self-weight: 640 $\frac{daN}{m^2}$;
- Permanent loads: 200 $\frac{daN}{m^2}$;
- Variable loads: 250 $\frac{daN}{m^2}$;

Steel grade is B450C: $f_{yd} = \frac{450}{1{,}15} = 391{,}3\ MPa$

Concrete is C25/30: $f_{cd} = \frac{0{,}85\ 25}{1{,}5} = 14{,}17\ MPa$

<u>BEAM R101: Verifications in phase I</u>

1. <u>Loads and internal actions in phase I</u>

   The steel truss in phase I has to resist alone to its self-weight (g_{truss}) , to the weight of the concrete casting ($g_{concrete}$) and to the weight (g_{slab}) from the influence area of the slab of 6 m about.

$$g_{truss} = 58\frac{daN}{m} = 0{,}0058\frac{kN}{m}$$

$$g_{concrete} = 937{,}5\frac{daN}{m} = 0{,}094\frac{kN}{m}$$

$$g_{slab} = 640\ \frac{daN}{m^2}\ 6\ m\ = 3840\frac{daN}{m} = 0{,}38\frac{kN}{m}$$

   The total load at ULS will be:

$$q_{ULS} = \ \gamma_G\ q = 1{,}3\ (0{,}0058 + 0{,}094 + 0{,}38) = 0{,}624\frac{kN}{m}$$

   For bending verification, the element is modelled as a simply supported beam of length 6m, the acting moment is:

$$M_{Sd} = \frac{q_{ULS}\ l^2}{8} = \frac{0{,}624\ 6^2}{8} = 2{,}8\ kNm$$

For shear verification is crucial to evaluate the stress in the single diagonal elements. The FEM model is implemented in the software “Lusas”: the steel reinforcement is modelled as “bar” element and a 3D analysis is carried out.

The distributed load q_{ULS} has been transformed into concentrated loads to be applied into the nodes of the truss:

$$F = q_{ULS}\, i = 0{,}624\ 0{,}4 = 0{,}25\ kN$$

In Figure 133 are shown the internal actions obtained into the model. The maximum shear force in the diagonal element is $V_{Sd,max} = 2{,}25\ kN$.

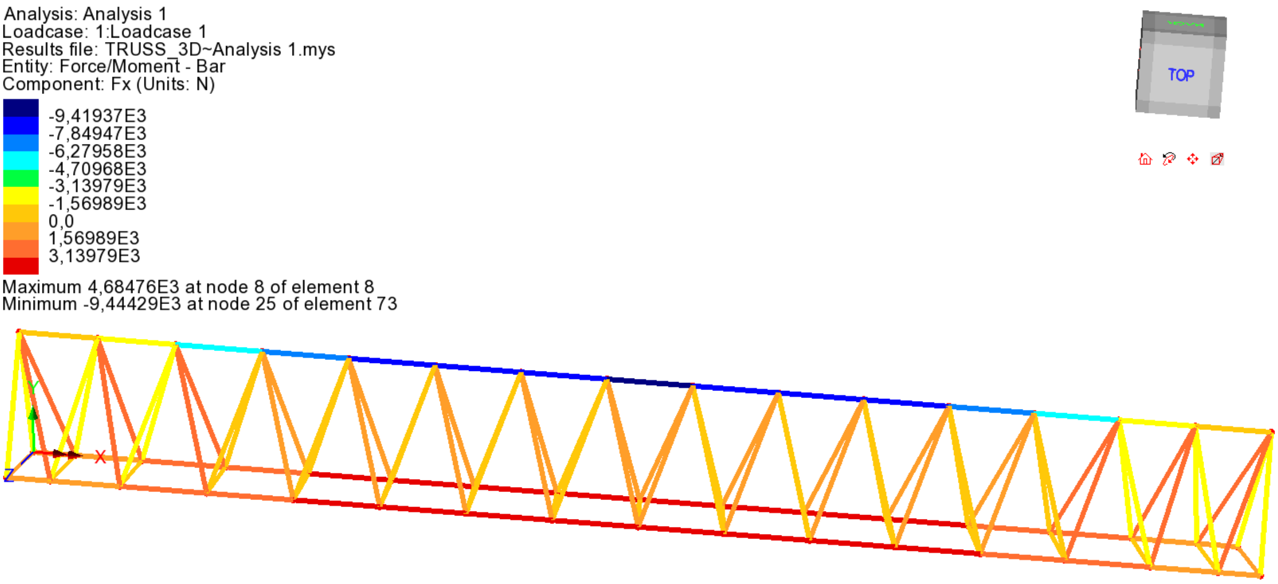

Figure 133 FEM model of the steel truss implemented in “Lusas”.

2. <u>Bending Resistances in phase I</u>
   The reinforcement layout given by the company is in Figure 134.

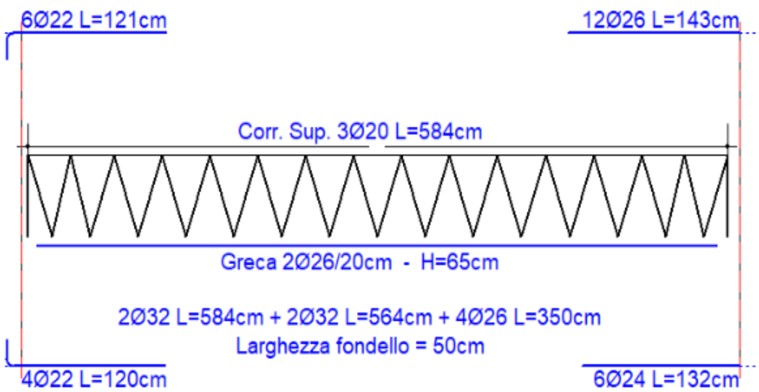

Figure 134 Reinforcement layout in beam R101.

The area of top reinforcement is $A_{sup} = 942\ mm^2$

The area of top reinforcement is $A_{inf} = 5340\ mm^2$

The area of the web is $A_{sw} = 1062\ mm^2$

The compression force in the top chord is given by:

$$C = A_{sup}\ f_{yd} = 368{,}6\ kN$$

The tension force in the bottom chord is given by:

$$T = A_{inf}\ f_{yd} = 368{,}6\ kN$$

The resisting bending moment is obtained as:

$$M_{rd} = \min(C, T)\ d = C\ d = 239{,}59\ kNm$$

3. Shear resistance in phase I
   The shear resistance of the entire truss depends on the resistance of the single element.
   The shear resistance is given by the web of the truss made by diagonal elements distributed with a triangular shape in the space with diameter ϕ26.
   Each diagonal element is axially loaded and exhibit a maximum resistance of:

$$V_{Rd} = \frac{A_{sw}\ f_{yd}}{\gamma_s} = \frac{\frac{1062}{2}\ 391{,}3}{1{,}15} = 180{,}6\ kN$$

BEAM R101: Verifications in phase II

1. Loads and internal actions in phase II
   In phase II the complete element has to resist alone to the loads of phase I and all the additional loads deriving from incidence of partitions, weight of screed, pavement and plaster, weight of fixed furniture and any additional load from the upper floors. The envelope of the internal actions has been evaluated through the use of the software DOLMEN and is clearly visible in Figure 128. From these envelopes the maximum internal actions are:

$$M^+_{Sd,max} = 1003{,}45\ kN$$
$$M^-_{Sd,max} = 1221{,}6\ kN$$
$$V_{Sd,max} = 1010{,}1\ kN$$

2. Bending Resistance in phase II
   The resisting bending moment is computed as:

$$M_{rd} = 0{,}9\ d\ f_{yd} A_s$$

   The value of the resisting bending moment vary along the beam due to the addition before the casting of the reinforcement for continuity purposes (figure 132).
   In midspan:

$$M^+_{rd} = 0{,}9\ d\ f_{yd} A_s = 0{,}9\ 650\ 391{,}3\ 5340 = 1222\ kNm$$
$$M^-_{rd} = 0{,}9\ d\ f_{yd} A_s = 0{,}9\ 650\ 391{,}3\ 942 = 215{,}63\ kNm$$

   In the initial section of the beam are added at the top 6 ϕ22 and at the bottom 4 ϕ22:

$$M_{rd}^{+} = 0{,}9\ d\ f_{yd} A_s = 0{,}9\ 650\ 391{,}3\ 1521 = 348\ kNm$$
$$M_{rd}^{-} = 0{,}9\ d\ f_{yd} A_s = 0{,}9\ 650\ 391{,}3\ 2281 = 522\ kNm$$

In the final section of the beam are added at the top 12 ϕ26 and at the bottom 6 ϕ24:

$$M_{rd}^{+} = 0{,}9\ d\ f_{yd} A_s = 0{,}9\ 650\ 391{,}3\ 2714 = 621\ kNm$$
$$M_{rd}^{-} = 0{,}9\ d\ f_{yd} A_s = 0{,}9\ 650\ 391{,}3\ 6371 = 1458{,}39\ kNm$$

The envelopes of bending moment are shown in Figure 135.

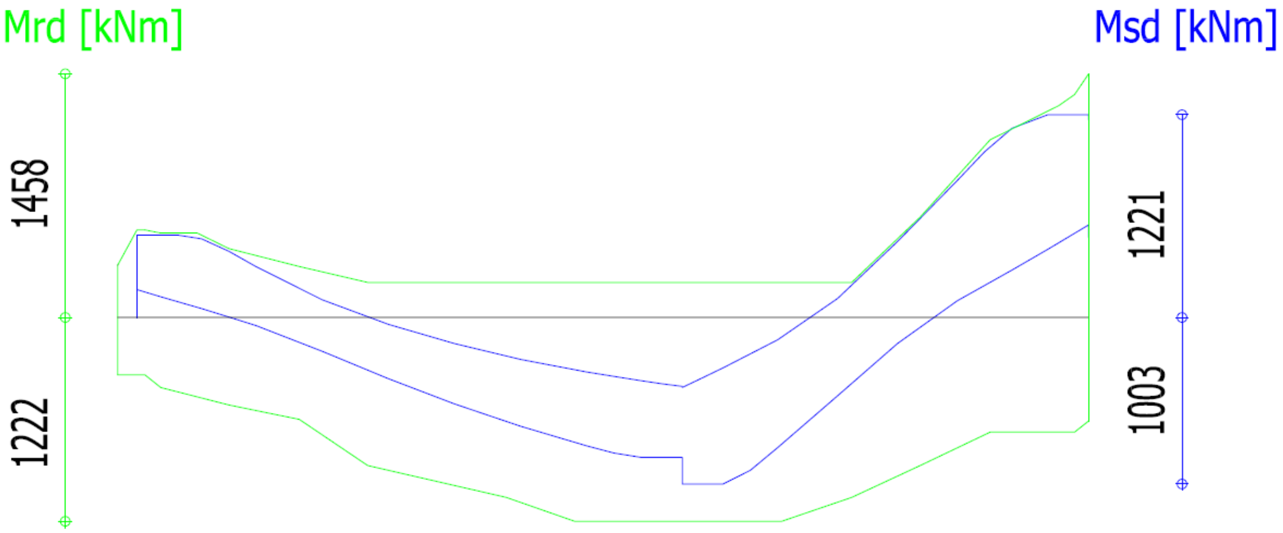

Figure 135 Envelopes of resisting bending moment (Mrd) and acting bending moment (Msd) for beam R101.

3. <u>Shear Resistance in phase II</u>

The computation of the shear resistance of the beam in phase II is carried following §4.1.2.3.5.2 of NTC2018 "Shear-resistant transverse reinforcement elements": the design resistance in shear V_{Rd} of structural elements with specific shear reinforcement must be evaluated on the basis of a suitable lattice scheme. The resistant elements of the ideal lattice are the transverse reinforcement, the longitudinal reinforcement, the compressed concrete stream and the inclined web struts.

The shear resistance takes as:

$$V_{Rd} = \min\left(V_{Rsd}, V_{Rcd}\right)$$

V_{Rsd} is the resistance on the side of steel, called in Italian "taglio - trazione" and is:

$$V_{Rsd} = 0{,}9\ d\ \frac{A_{sw}}{s}\ f_{yd}(\cot\alpha + \cot\theta)\sin\alpha$$

V_{Rcd} is the resistance on the side of concrete, called in Italian "taglio - compressione" and is:

$$V_{Rcd} = 0{,}9\ d\ b_w \alpha_c\ v\ f_{cd}\ \frac{(\cot\alpha + \cot\theta)}{(1 + \cot\theta^2)}$$

where:

d is the clear height of the section;

s is the spacing between consecutive transversal reinforcement;

A_{sw} is the area of transversal reinforcement;

v is a reduction factor equal to 0,5;

α is the inclination angle of the transversal reinforcement respect to the beam axis;

θ is the inclination of the compressed concrete streams respect to the beam axis and must be $1 \leq \cot\theta \leq 2{,}5$;

b_w is the minimum width of the section;

α_c is qual to 1 in non-prestressed structures.

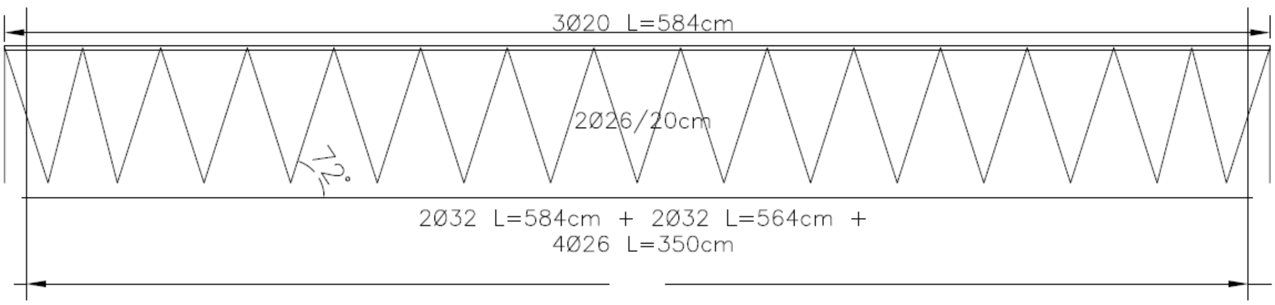

Figure 136 Layout of the beam R101.

In the current case, beam R101 (Figure 136), the section is 50 x 75 cm, s = 20 cm, α = 72° and θ=45° in order to get $\cot\theta$=1 and minimise the resistance.

$$V_{Rsd} = 0{,}9\ d\ \frac{A_{sw}}{s}\ f_{yd}(\cot\alpha + \cot\theta)\sin\alpha = 0{,}9\ 650\ \frac{1062}{200}\ 391{,}3\ (0{,}32 + 1)0{,}95 = 1525\ kN$$

$$V_{Rcd} = 0{,}9\ d\ b_w \alpha_c\ v\ f_{cd}\ \frac{(\cot\alpha + \cot\theta)}{(1+\cot\theta^2)} = 0{,}9\ 650\ 500\ 1\ 0{,}5\ 14{,}1\ \frac{0{,}32+1}{1+1} = 1361\ kN$$

$$V_{Rd} = \min(V_{Rsd}, V_{Rcd}) = V_{Rcd} = 1361\ kN$$

The envelopes of shear are shown in Figure 137.

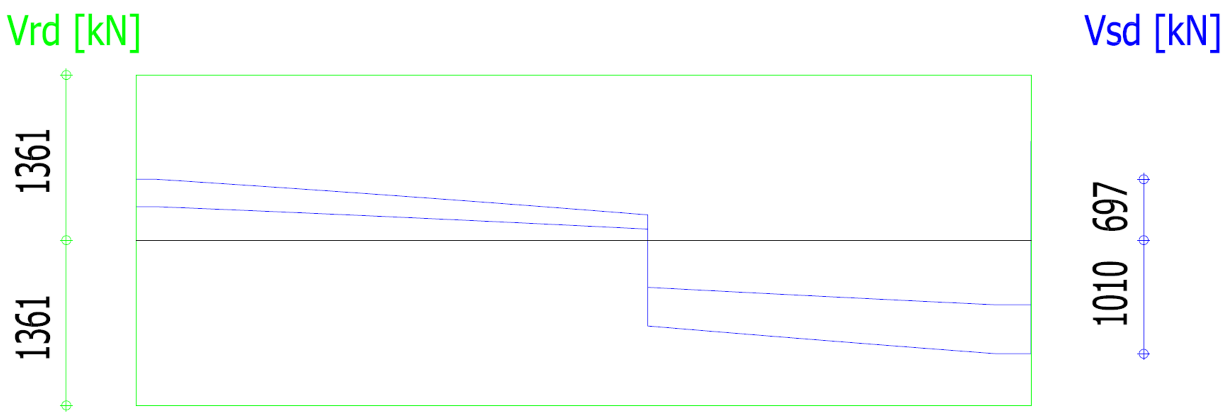

Figure 137 Envelopes of resisting shear (Vrd) and acting shear (Vsd) for beam R101.

BEAM R102: Verifications in phase I

1. Loads and internal actions in phase I

   The steel truss in phase I has to resist alone to its self-weight (g_{truss}) , to the weight of the concrete casting ($g_{concrete}$) and to the weight (g_{slab}) from the influence area of the slab of 6 m about.

$$g_{truss} = 55,84 \frac{daN}{m} = 0,0056 \frac{kN}{m}$$

$$g_{concrete} = 937,5 \frac{daN}{m} = 0,094 \frac{kN}{m}$$

$$g_{slab} = 640 \; \frac{daN}{m^2} \; 6 \, m \; = 3840 \frac{daN}{m} = 0,38 \frac{kN}{m}$$

The total load at ULS will be:

$$q_{ULS} = \gamma_G \, q = 1,3 \; (0,0056 + 0,094 + 0,38) = 0,624 \frac{kN}{m}$$

For bending verification, the element is modelled as a simply supported beam of length 6,1 m, the acting moment is:

$$M_{Sd} = \frac{q_{ULS} \; l^2}{8} = \frac{0,624 \; 6,1^2}{8} = 2,9 \; kNm$$

For shear verification is crucial to evaluate the stress in the single diagonal elements. The FEM model is implemented in the software “Lusas”: the steel reinforcement is modelled as “bar” element and a 3D analysis is carried out.

The distributed load q_{ULS} has been transformed into concentrated loads to be applied into the nodes of the truss:

$$F = \; q_{ULS} \; i = 0,624 \; 0,4 = 0,25 \; kN$$

In Figure 138 are shown the internal actions obtained into the model. The maximum shear force in the diagonal element is $V_{Sd,max} \; = 2,5 \; kN$.

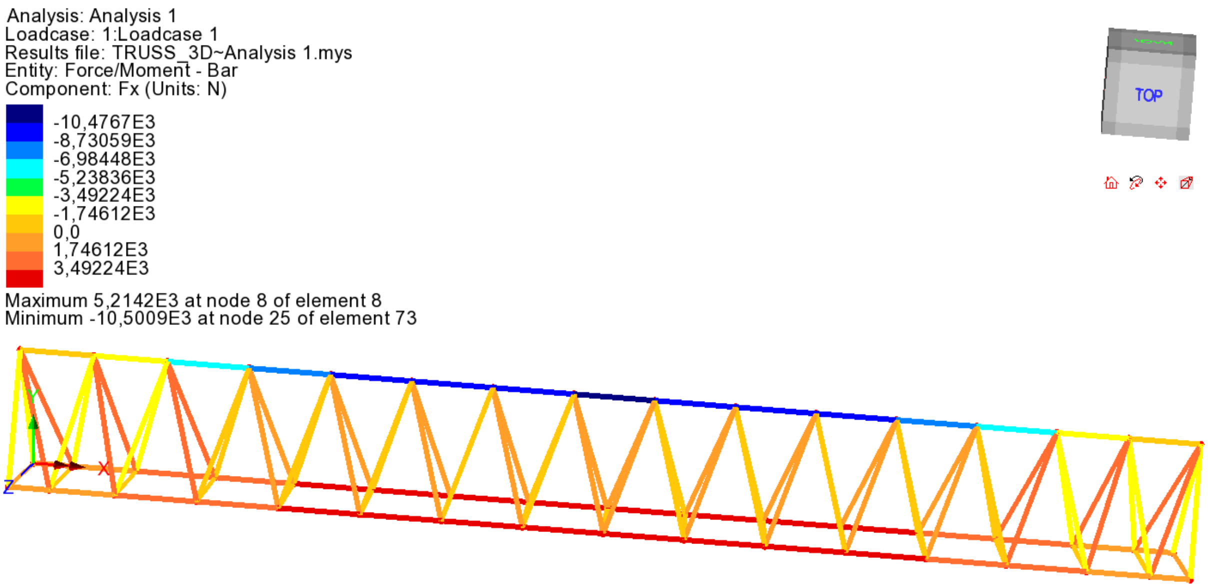

Figure 138 FEM model of the steel truss implemented in "Lusas".

2. <u>Bending Resistances in phase I</u>

   The reinforcement layout given by the company is in Figure 139.

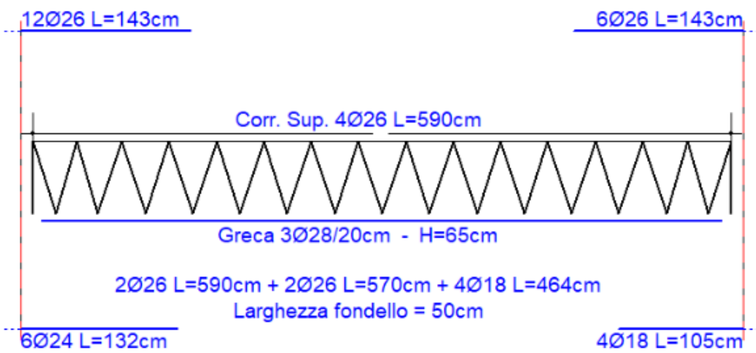

Figure 139 Reinforcement layout in beam R102.

The area of top reinforcement is $A_{sup} = 2124\ mm^2$

The area of top reinforcement is $A_{inf} = 3142\ mm^2$

The area of the web is $A_{sw} = 1847\ mm^2$

The compression force in the top chord is given by:

$$C = A_{sup}\, f_{yd} = 831\ kN$$

The tension force in the bottom chord is given by:

$$T = A_{inf}\, f_{yd} = 1229{,}5\ kN$$

The resisting bending moment is obtained as:

$$M_{rd} = \min(C, T)\ d = C\ d = 540{,}15\ kNm$$

3. <u>Shear resistance in phase I</u>
   The shear resistance of the entire truss depends on the resistance of the single element.
   The shear resistance is given by the web of the truss made by diagonal elements distributed with a triangular shape in the space with diameter ϕ28.
   Each diagonal element is axially loaded and exhibit a maximum resistance of:

$$V_{Rd} = \frac{A_{sw}\ f_{yd}}{\gamma_s} = \frac{\frac{1847}{2}\ 391{,}3}{1{,}15} = 314{,}2\ kN$$

<u>BEAM R102: Verifications in phase II</u>

1. <u>Loads and internal actions in phase II</u>
   In phase II the complete element has to resist alone to the loads of phase I and all the additional loads deriving from incidence of partitions, weight of screed, pavement and plaster, weight of fixed furniture and any additional load from the upper floors. The envelope of the internal actions has been evaluated through the use of the software DOLMEN and is clearly visible in Figure 128. From these envelopes the maximum internal actions are:

$$M^{+}_{Sd,max} = 497{,}15\ kN$$
$$M^{-}_{Sd,max} = 1221{,}6\ kN$$
$$V_{Sd,max} = 1100{,}1\ kN$$

2. <u>Bending Resistance in phase II</u>
   The resisting bending moment is computed as:

$$M_{rd} = 0{,}9\ d\ f_{yd} A_s$$

   The value of the resisting bending moment vary along the beam due to the addition before the casting of the reinforcement for continuity purposes.
   In midspan:

$$M^{+}_{rd} = 0{,}9\ d\ f_{yd} A_s = 0{,}9\ 650\ 391{,}3\ 3142 = 719\ kNm$$
$$M^{-}_{rd} = 0{,}9\ d\ f_{yd} A_s = 0{,}9\ 650\ 391{,}3\ 2124 = 486\ kNm$$

   In the initial section of the beam are added at the top 12 ϕ26 and at the bottom 6 ϕ24:

$$M^{+}_{rd} = 0{,}9\ d\ f_{yd} A_s = 0{,}9\ 650\ 391{,}3\ 2714 = 621\ kNm$$
$$M^{-}_{rd} = 0{,}9\ d\ f_{yd} A_s = 0{,}9\ 650\ 391{,}3\ 6371 = 1458\ kNm$$

   In the final section of the beam are added at the top 6 ϕ26 and at the bottom 4 ϕ18:

$$M^{+}_{rd} = 0{,}9\ d\ f_{yd} A_s = 0{,}9\ 650\ 391{,}3\ 1018 = 233\ kNm$$
$$M^{-}_{rd} = 0{,}9\ d\ f_{yd} A_s = 0{,}9\ 650\ 391{,}3\ 3186 = 729\ kNm$$

   The envelopes of bending moment are illustrated in Figure 140.

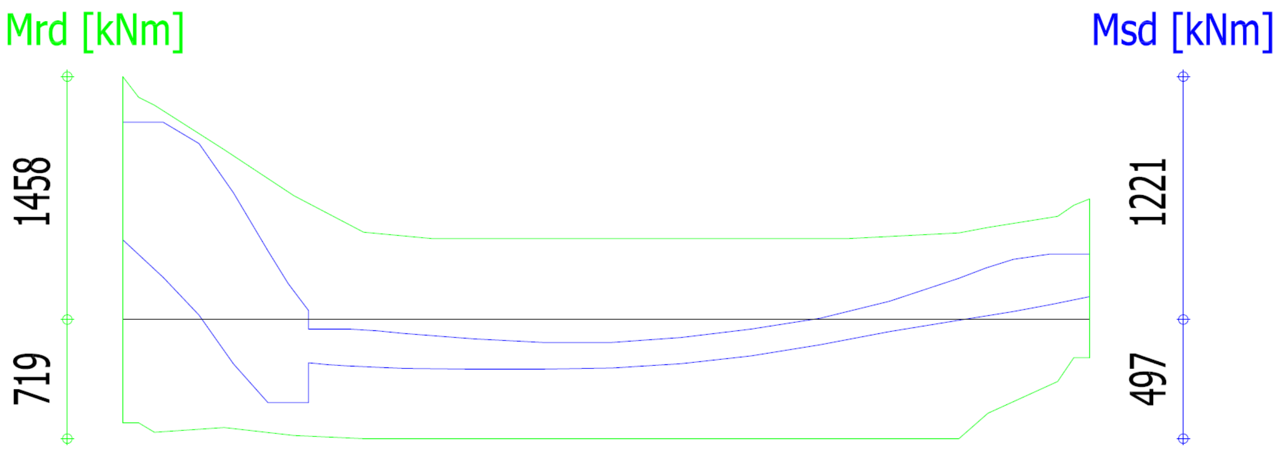

Figure 140 Envelopes of resisting bending moment (Mrd) and acting bending moment (Msd) for beam R102.

3. <u>Shear Resistance in phase II</u>
The computation of the shear resistance of the beam in phase II is carried following §4.1.2.3.5.2 of NTC2018 “Shear-resistant transverse reinforcement elements”: the design resistance in shear V_{Rd} of structural elements with specific shear reinforcement must be evaluated on the basis of a suitable lattice scheme. The resistant elements of the ideal lattice are the transverse reinforcement, the longitudinal reinforcement, the compressed concrete stream and the inclined web struts.
The shear resistance takes as:

$$V_{Rd} = \min\,(V_{Rsd}, V_{Rcd})$$

V_{Rsd} is the resistance on the side of steel, called in Italian “taglio - trazione” and is:

$$V_{Rsd} = 0{,}9\,d\,\frac{A_{sw}}{s}\,f_{yd}(\cot\alpha + \cot\theta)\sin\alpha$$

V_{Rcd} is the resistance on the side of concrete, called in Italian “taglio - compressione” and is:

$$V_{Rcd} = 0{,}9\,d\,b_w\alpha_c\,v\,f_{cd}\,\frac{(\cot\alpha + \cot\theta)}{(1+\cot\theta^2)}$$

where:

d is the clear height of the section;

s is the spacing between consecutive transversal reinforcement;

A_{sw} is the area of transversal reinforcement;

v is a reduction factor equal to 0,5;

α is the inclination angle of the transversal reinforcement respect to the beam axis;

θ is the inclination of the compressed concrete streams respect to the beam axis and must be $1 \leq \cot\theta \leq 2{,}5$;

b_w is the minimum width of the section;

α_c is qual to 1 in non-prestressed structures.

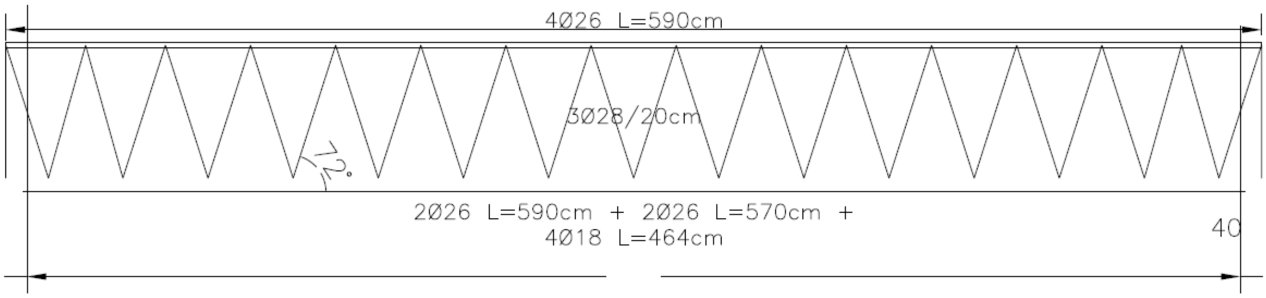

Figure 141 Layout of the beam R102.

In the current case, beam R102 (Figure 141), the section is 50 x 75 cm, s = 20 cm, α = 72° and θ=45° in order to get $\cot\theta$=1 and minimise the resistance.

$$V_{Rsd} = 0{,}9\, d\, \frac{A_{sw}}{s}\, f_{yd}(\cot\alpha + \cot\theta)\sin\alpha = 0{,}9\ 650\ \frac{1847}{200}\ 391{,}3\ (0{,}32 + 1)0{,}95 = 2654\ kN$$

$$V_{Rcd} = 0{,}9\, d\, b_w \alpha_c\, v\, f_{cd}\, \frac{(\cot\alpha + \cot\theta)}{(1 + \cot\theta^2)} = 0{,}9\ 650\ 500\ 1\ 0{,}5\ 14{,}1\ \frac{0{,}32+1}{1+1} = 1361\ kN$$

$$V_{Rd} = \min(V_{Rsd}, V_{Rcd}) = V_{Rcd} = 1361\ kN$$

The envelopes of shear are shown in Figure 142.

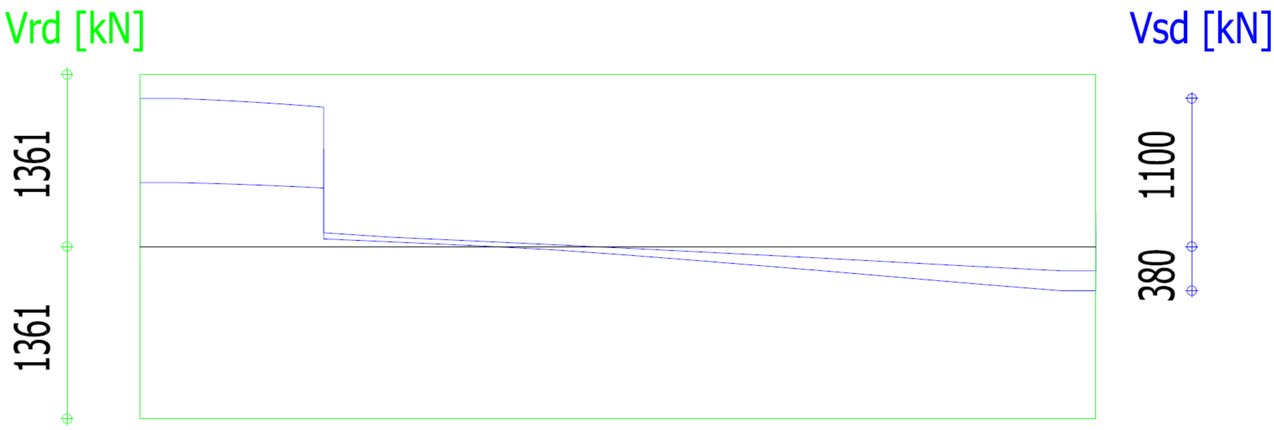

Figure 142 Envelopes of resisting shear (Vrd) and acting shear (Vsd) for beam R102.

BEAM R103: Verifications in phase I

1. Loads and internal actions in phase I

   The steel truss in phase I has to resist alone to its self-weight (g_{truss}) , to the weight of the concrete casting ($g_{concrete}$) and to the weight (g_{slab}) from the influence area of the slab of 6 m about.

$$g_{truss} = 15{,}68 \frac{daN}{m} = 0{,}0016 \frac{kN}{m}$$

$$g_{concrete} = 687{,}5 \frac{daN}{m} = 0{,}069 \frac{kN}{m}$$

$$g_{slab} = 640 \ \frac{daN}{m^2} \ 6 \ m \ = 3840 \frac{daN}{m} = 0{,}38 \frac{kN}{m}$$

The total load at ULS will be:

$$q_{ULS} = \ \gamma_G \ q = 1{,}3 \ (0{,}0016 + 0{,}069 + 0{,}38) = 0{,}59 \frac{kN}{m}$$

For bending verification, the element is modelled as a simply supported beam of length 5,18 m, the acting moment is:

$$M_{Sd} = \ \frac{q_{ULS} \ l^2}{8} = \frac{0{,}59 \ 5{,}18^2}{8} = 1{,}99 \ kNm$$

For shear verification is crucial to evaluate the stress in the single diagonal elements. The FEM model is implemented in the software "Lusas": the steel reinforcement is modelled as "bar" element and a 3D analysis is carried out.

The distributed load q_{ULS} has been transformed into concentrated loads to be applied into the nodes of the truss:

$$F = \ q_{ULS} \ i = 0{,}59 \ 0{,}4 = 0{,}24 \ kN$$

In Figure 143 are shown the internal actions obtained into the model. The maximum shear force in the diagonal element is $V_{Sd,max} \ = 1{,}18 \ kN$.

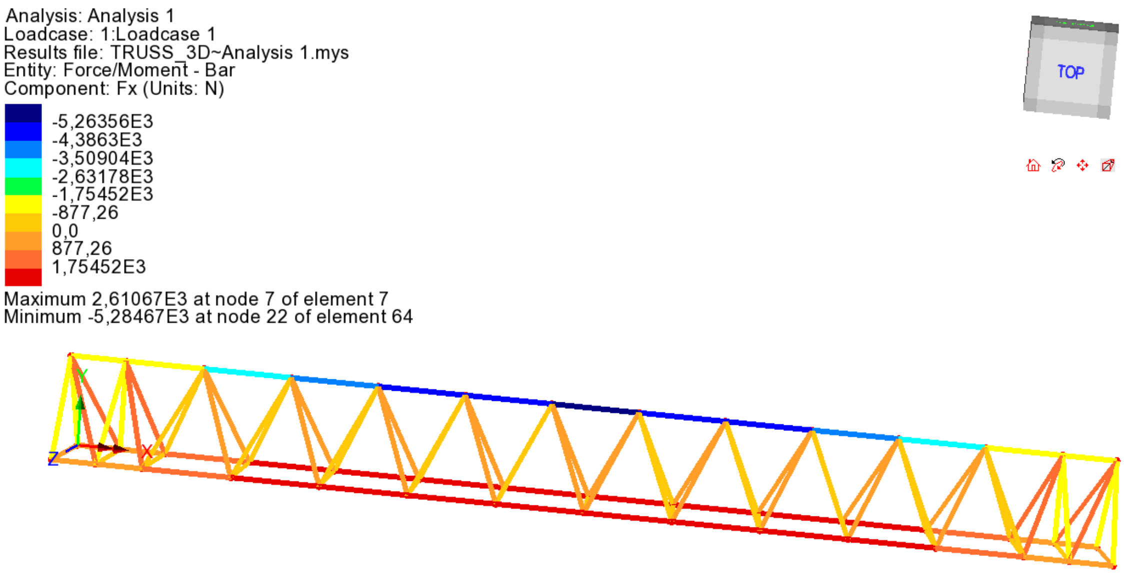

Figure 143 FEM model of the steel truss implemented in "Lusas".

2. <u>Bending Resistances in phase I</u>

The reinforcement layout given by the company is in Figure 144.

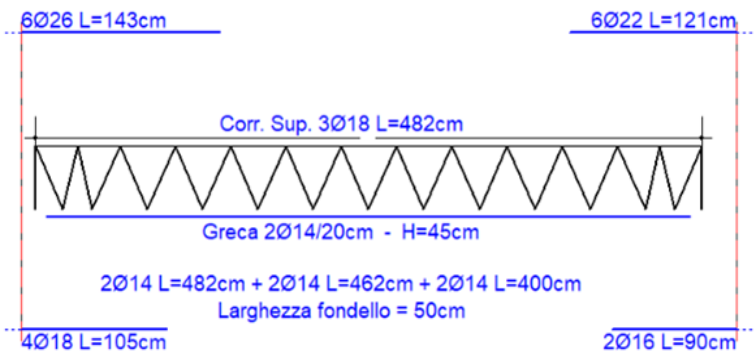

Figure 144 Reinforcement layout in beam R103.

The area of top reinforcement is $A_{sup} = 763\ mm^2$

The area of top reinforcement is $A_{inf} = 924\ mm^2$

The area of the web is $A_{sw} = 308\ mm^2$

The compression force in the top chord is given by:

$$C = A_{sup}\, f_{yd} = 298{,}6\ kN$$

The tension force in the bottom chord is given by:

$$T = A_{inf}\, f_{yd} = 361{,}5\ kN$$

The resisting bending moment is obtained as:

$$M_{rd} = \min(C, T)\ d = C\ d = 134{,}4\ kNm$$

3. Shear resistance in phase I
The shear resistance of the entire truss depends on the resistance of the single element. The shear resistance is given by the web of the truss made by diagonal elements distributed with a triangular shape in the space with diameter ϕ14.
Each diagonal element is axially loaded and exhibit a maximum resistance of:

$$V_{Rd} = \frac{A_{sw}\, f_{yd}}{\gamma_s} = \frac{\frac{308}{2}\ 391{,}3}{1{,}15} = 52{,}4\ kN$$

BEAM R103: Verifications in phase II

1. Loads and internal actions in phase II
In phase II the complete element has to resist alone to the loads of phase I and all the additional loads deriving from incidence of partitions, weight of screed, pavement and plaster, weight of fixed furniture and any additional load from the upper floors. The envelope of the internal actions has been evaluated through the use of the software DOLMEN and is clearly visible in Figure 128. From these envelopes the maximum internal actions are:

$$M^{+}_{Sd,max} = 139\ kN$$
$$M^{-}_{Sd,max} = 392\ kN$$
$$V_{Sd,max} = 299{,}1\ kN$$

2. Bending Resistance in phase II
The resisting bending moment is computed as:

$$M_{rd} = 0{,}9\ d\ f_{yd} A_s$$

The value of the resisting bending moment vary along the beam due to the addition before the casting of the reinforcement for continuity purposes.
In midspan:

$$M^{+}_{rd} = 0{,}9\ d\ f_{yd} A_s = 0{,}9\ 450\ 391{,}3\ 924 = 146\ kNm$$
$$M^{-}_{rd} = 0{,}9\ d\ f_{yd} A_s = 0{,}9\ 450\ 391{,}3\ 763 = 121\ kNm$$

In the initial section of the beam are added at the top 6 ϕ26 and at the bottom 4 ϕ18:

$$M^{+}_{rd} = 0{,}9\ d\ f_{yd} A_s = 0{,}9\ 450\ 391{,}3\ 1018 = 161\ kNm$$
$$M^{-}_{rd} = 0{,}9\ d\ f_{yd} A_s = 0{,}9\ 450\ 391{,}3\ 3186 = 504\ kNm$$

In the final section of the beam are added at the top 6 ϕ22 and at the bottom 2 ϕ16:

$$M^{+}_{rd} = 0{,}9\ d\ f_{yd} A_s = 0{,}9\ 450\ 391{,}3\ 402 = 64\ kNm$$
$$M^{-}_{rd} = 0{,}9\ d\ f_{yd} A_s = 0{,}9\ 450\ 391{,}3\ 2281 = 361\ kNm$$

The envelopes of bending moment are illustrated in Figure 145.

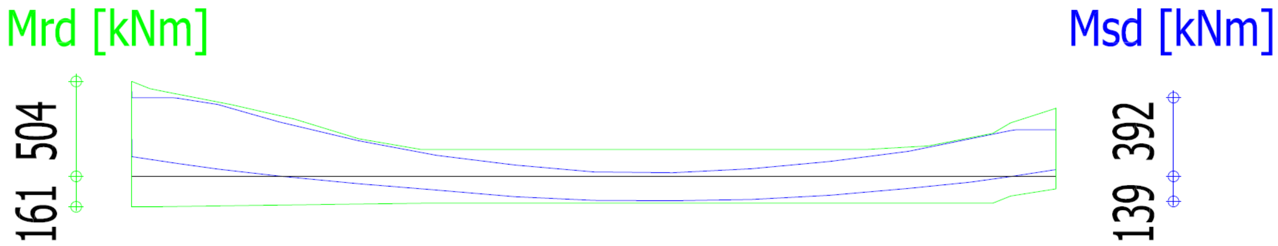

Figure 145 Envelopes of resisting bending moment (Mrd) and acting bending moment (Msd) for beam R103.

3. Shear Resistance in phase II

The computation of the shear resistance of the beam in phase II is carried following §4.1.2.3.5.2 of NTC2018 "Shear-resistant transverse reinforcement elements": the design resistance in shear V_{Rd} of structural elements with specific shear reinforcement must be evaluated on the basis of a suitable lattice scheme. The resistant elements of the ideal lattice are the transverse reinforcement, the longitudinal reinforcement, the compressed concrete stream and the inclined web struts.

The shear resistance takes as:

$$V_{Rd} = \min\,(V_{Rsd}, V_{Rcd})$$

V_{Rsd} is the resistance on the side of steel, called in Italian "taglio - trazione" and is:

$$V_{Rsd} = 0{,}9\; d\; \frac{A_{sw}}{s}\; f_{yd}(\cot\alpha + \cot\theta)\sin\alpha$$

V_{Rcd} is the resistance on the side of concrete, called in Italian "taglio - compressione" and is:

$$V_{Rcd} = 0{,}9\; d\; b_w \alpha_c\; v\; f_{cd}\; \frac{(\cot\alpha + \cot\theta)}{(1 + \cot\theta^2)}$$

where:

d is the clear height of the section;

s is the spacing between consecutive transversal reinforcement;

A_{sw} is the area of transversal reinforcement;

v is a reduction factor equal to 0,5;

α is the inclination angle of the transversal reinforcement respect to the beam axis;

θ is the inclination of the compressed concrete streams respect to the beam axis and must be $1 \leq \cot\theta \leq 2{,}5$;

b_w is the minimum width of the section;

α_c is qual to 1 in non-prestressed structures.

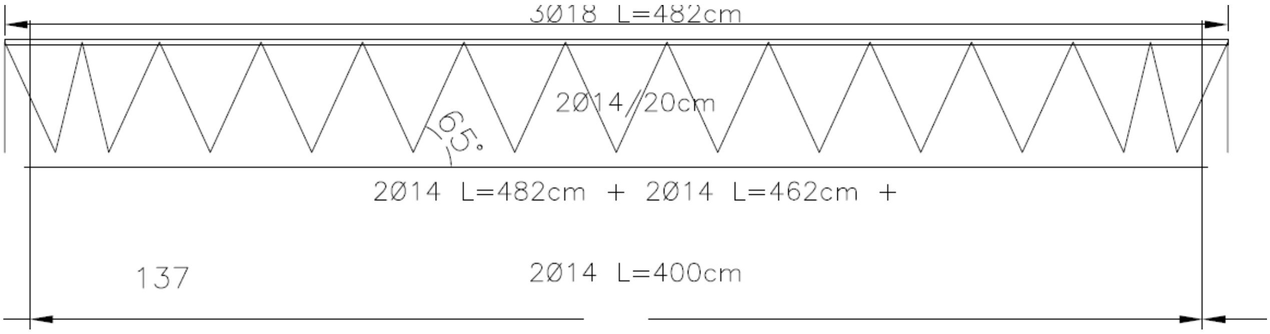

Figure 146 Layout of the beam R103.

In the current case, beam R103 (Figure 146), the section is 50 x 55 cm, s = 20 cm, α = 65° and θ=45° in order to get cot θ=1 and minimise the resistance.

$$V_{Rsd} = 0{,}9\, d\, \frac{A_{sw}}{s}\, f_{yd}(\cot\alpha + \cot\theta)\sin\alpha = 0{,}9\ 450\ \frac{308}{200}\ 391{,}3\ (0{,}466 + 1)0{,}906 = 324\ kN$$

$$V_{Rcd} = 0{,}9\, d\, b_w \alpha_c\, v\, f_{cd}\, \frac{(\cot\alpha+\cot\theta)}{(1+\cot\theta^2)} = 0{,}9\ 450\ 500\ 1\ 0{,}5\ 14{,}1\ \frac{0{,}466+1}{1+1} = 1046\ kN$$

$$V_{Rd} = \min(V_{Rsd}, V_{Rcd}) = V_{RSd} = 324\ kN$$

The envelopes of shear are shown in Figure 147.

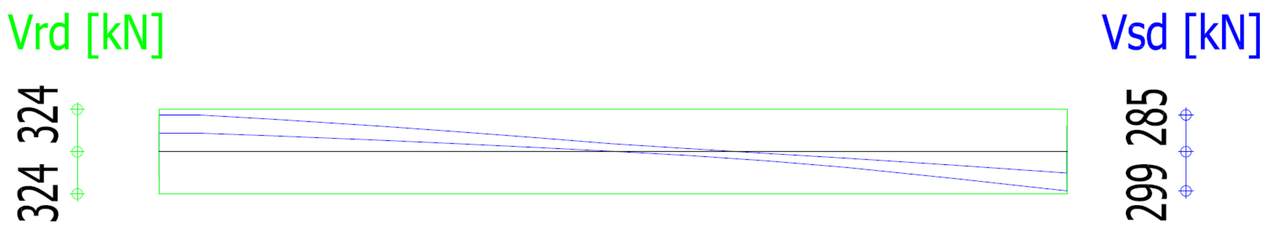

Figure 147 Envelopes of resisting shear (Vrd) and acting shear (Vsd) for beam R103.

BEAM R104: Verifications in phase I

1. Loads and internal actions in phase I

   The steel truss in phase I has to resist alone to its self-weight (g_{truss}) , to the weight of the concrete casting ($g_{concrete}$) and to the weight (g_{slab}) from the influence area of the slab of 6 m about.

$$g_{truss} = 18 \frac{daN}{m} = 0{,}0018 \frac{kN}{m}$$

$$g_{concrete} = 687{,}5 \frac{daN}{m} = 0{,}069 \frac{kN}{m}$$

$$g_{slab} = 640 \ \frac{daN}{m^2} \ 6 \ m \ = 3840 \frac{daN}{m} = 0{,}38 \frac{kN}{m}$$

The total load at ULS will be:

$$q_{ULS} = \ \gamma_G \ q = 1{,}3 \ (0{,}0018 + 0{,}069 + 0{,}38) = 0{,}59 \frac{kN}{m}$$

For bending verification, the element is modelled as a simply supported beam of length 5,59 m, the acting moment is:

$$M_{Sd} = \frac{q_{ULS} \ l^2}{8} = \frac{0{,}59 \ 5{,}59^2}{8} = 2{,}3 \ kNm$$

For shear verification is crucial to evaluate the stress in the single diagonal elements. The FEM model is implemented in the software “Lusas”: the steel reinforcement is modelled as “bar” element and a 3D analysis is carried out.

The distributed load q_{ULS} has been transformed into concentrated loads to be applied into the nodes of the truss:

$$F = \ q_{ULS} \ i = 0{,}59 \ 0{,}4 = 0{,}24 \ kN$$

In Figure 148 are shown the internal actions obtained into the model. The maximum shear force in the diagonal element is $V_{Sd,max} \ = 1{,}299 \ kN$.

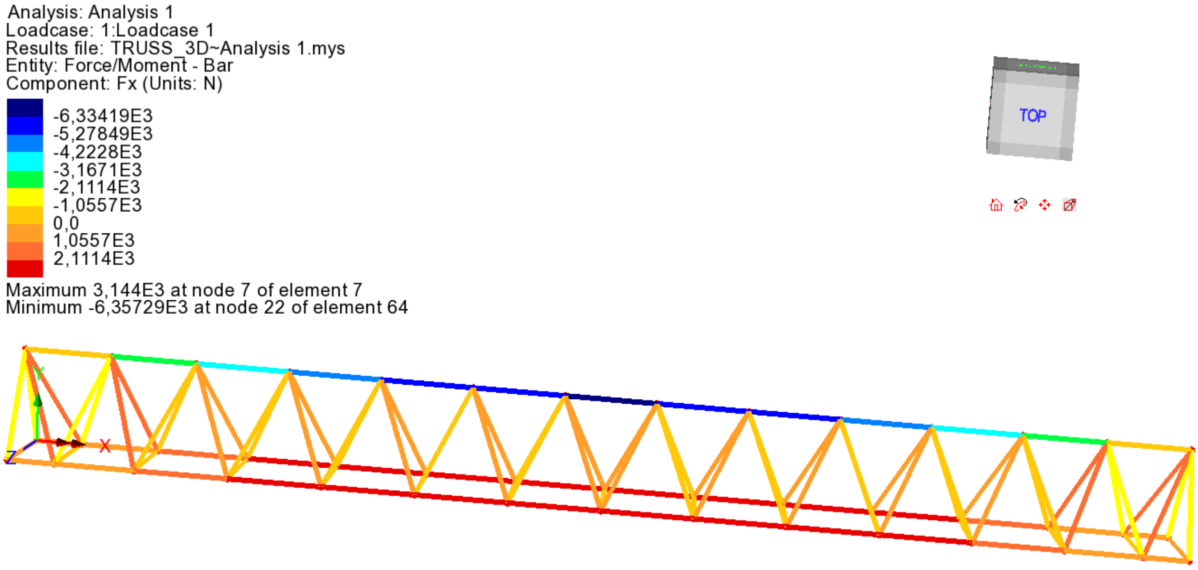

Figure 148 FEM model of the steel truss implemented in "Lusas".

2. <u>Bending Resistances in phase I</u>

   The reinforcement layout given by the company is in Figure 149.

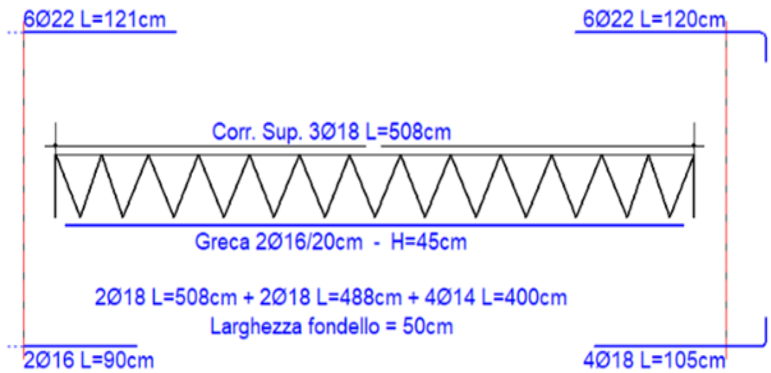

Figure 149 Reinforcement layout in beam R104.

The area of top reinforcement is $A_{sup} = 763\ mm^2$

The area of top reinforcement is $A_{inf} = 1634\ mm^2$

The area of the web is $A_{sw} = 402\ mm^2$

The compression force in the top chord is given by:

$$C = A_{sup}\, f_{yd} = 298{,}6\ kN$$

The tension force in the bottom chord is given by:

$$T = A_{inf}\, f_{yd} = 639{,}4\ kN$$

The resisting bending moment is obtained as:

$$M_{rd} = \min(C,T)\ d = C\ d = 134{,}4\ kNm$$

3. <u>Shear resistance in phase I</u>
   The shear resistance of the entire truss depends on the resistance of the single element. The shear resistance is given by the web of the truss made by diagonal elements distributed with a triangular shape in the space with diameter ϕ16.
   Each diagonal element is axially loaded and exhibit a maximum resistance of:

$$V_{Rd} = \frac{A_{sw}\, f_{yd}}{\gamma_s} = \frac{\frac{402}{2}\ 391{,}3}{1{,}15} = 68{,}4\ kN$$

<u>BEAM R104: Verifications in phase II</u>

1. <u>Loads and internal actions in phase II</u>
   In phase II the complete element has to resist alone to the loads of phase I and all the additional loads deriving from incidence of partitions, weight of screed, pavement and plaster, weight of fixed furniture and any additional load from the upper floors. The envelope of the internal actions has been evaluated through the use of the software DOLMEN and is clearly visible in Figure 128. From these envelopes the maximum internal actions are:

$$M^{+}_{Sd,max} = 244\ kN$$
$$M^{-}_{Sd,max} = 306\ kN$$
$$V_{Sd,max} = 353\ kN$$

2. <u>Bending Resistance in phase II</u>
   The resisting bending moment is computed as:

$$M_{rd} = 0{,}9\ d\ f_{yd} A_s$$

   The value of the resisting bending moment vary along the beam due to the addition before the casting of the reinforcement for continuity purposes.
   In midspan:

$$M^{+}_{rd} = 0{,}9\ d\ f_{yd} A_s = 0{,}9\ 450\ 391{,}3\ 1634 = 259\ kNm$$
$$M^{-}_{rd} = 0{,}9\ d\ f_{yd} A_s = 0{,}9\ 450\ 391{,}3\ 763 = 121\ kNm$$

   In the initial section of the beam are added at the top 6 ϕ22 and at the bottom 2 ϕ16:

$$M^{+}_{rd} = 0{,}9\ d\ f_{yd} A_s = 0{,}9\ 450\ 391{,}3\ 402 = 64\ kNm$$
$$M^{-}_{rd} = 0{,}9\ d\ f_{yd} A_s = 0{,}9\ 450\ 391{,}3\ 2281 = 361\ kNm$$

   In the final section of the beam are added at the top 6 ϕ22 and at the bottom 4 ϕ18:

$$M^{+}_{rd} = 0{,}9\ d\ f_{yd} A_s = 0{,}9\ 450\ 391{,}3\ 1018 = 161\ kNm$$
$$M^{-}_{rd} = 0{,}9\ d\ f_{yd} A_s = 0{,}9\ 450\ 391{,}3\ 2281 = 361\ kNm$$

   The envelopes of bending moment are showed in Figure 150.

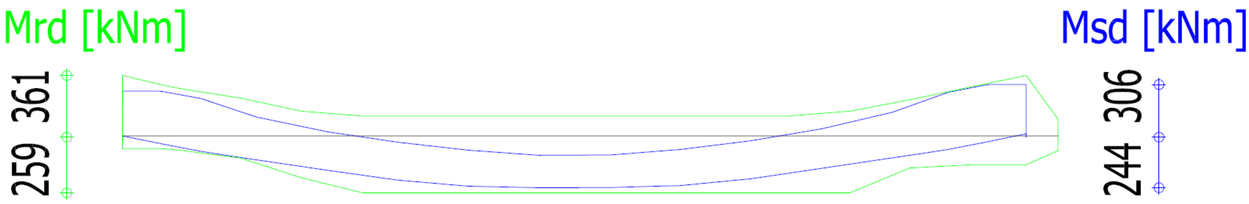

Figure 150 Envelopes of resisting bending moment (Mrd) and acting bending moment (Msd) for beam R104.

3. <u>Shear Resistance in phase II</u>

The computation of the shear resistance of the beam in phase II is carried following §4.1.2.3.5.2 of NTC2018 "Shear-resistant transverse reinforcement elements": the design resistance in shear V_{Rd} of structural elements with specific shear reinforcement must be evaluated on the basis of a suitable lattice scheme. The resistant elements of the ideal lattice are the transverse reinforcement, the longitudinal reinforcement, the compressed concrete stream and the inclined web struts.

The shear resistance takes as:

$$V_{Rd} = \min\,(V_{Rsd}, V_{Rcd})$$

V_{Rsd} is the resistance on the side of steel, called in Italian "taglio - trazione" and is:

$$V_{Rsd} = 0{,}9\ d\ \frac{A_{sw}}{s}\ f_{yd}(\cot\alpha + \cot\theta)\sin\alpha$$

V_{Rcd} is the resistance on the side of concrete, called in Italian "taglio - compressione" and is:

$$V_{Rcd} = 0{,}9\ d\ b_w \alpha_c\ v\ f_{cd}\ \frac{(\cot\alpha + \cot\theta)}{(1 + \cot\theta^2)}$$

where:

d is the clear height of the section;

s is the spacing between consecutive transversal reinforcement;

A_{sw} is the area of transversal reinforcement;

v is a reduction factor equal to 0,5;

α is the inclination angle of the transversal reinforcement respect to the beam axis;

θ is the inclination of the compressed concrete streams respect to the beam axis and must be $1 \le \cot\theta \le 2{,}5$;

b_w is the minimum width of the section;

α_c is qual to 1 in non-prestressed structures.

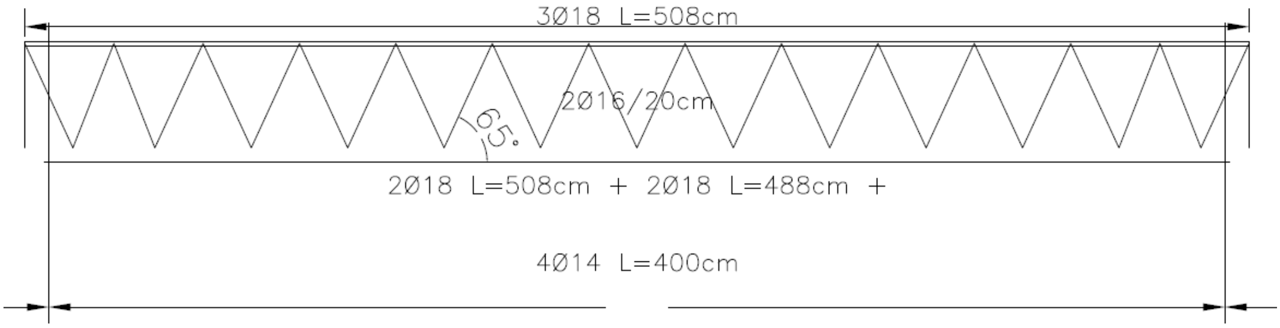

Figure 151 Layout of the beam R104.

In the current case, beam R103 (Figure 151), the section is 50 x 55 cm, s = 20 cm, α = 65° and θ=45° in order to get cot θ=1 and minimise the resistance.

$$V_{Rsd} = 0{,}9\ d\ \frac{A_{sw}}{s}\ f_{yd}(\cot\alpha + \cot\theta)\sin\alpha = 0{,}9\ 450\ \frac{402}{200}\ 391{,}3\ (0{,}466 + 1)0{,}906 = 423\ kN$$

$$V_{Rcd} = 0{,}9\ d\ b_w \alpha_c\ v\ f_{cd}\ \frac{(\cot\alpha + \cot\theta)}{(1+\cot\theta^2)} = 0{,}9\ 450\ 500\ 1\ 0{,}5\ 14{,}1\ \frac{0{,}466+1}{1+1} = 1046\ kN$$

$$V_{Rd} = \min(V_{Rsd}, V_{Rcd}) = V_{RSd} = 423\ kN$$

The envelopes of shear are shown in Figure 152.

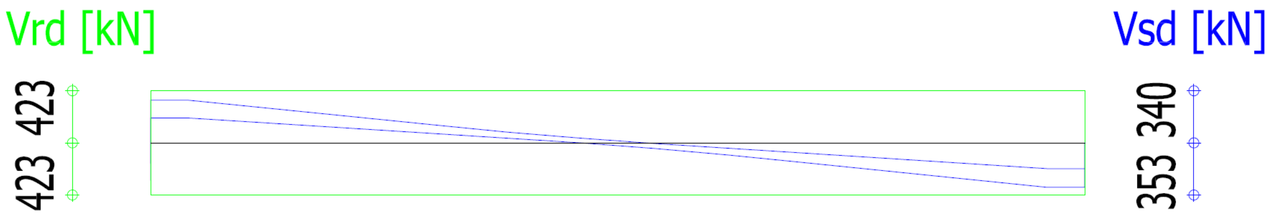

Figure 152 Envelopes of resisting shear (Vrd) and acting shear (Vsd) for beam R104.

To conclude is possible to observe that all the verifications, both in phase I and in phase II, are satisfied.

In phase I the truss seems over-dimensioned. The main reason is that the huge capacity, derived from its geometry and configuration of rebars, is not fully necessary in phase I but is needed in phase II when also the additional loads will be considered, in particular those of pillars P22 and P25 (scheme of beam supporting columns) in the heavily loaded spans of T14 (R101 and R102).

4.5 APPLICATION OF CSTCBs IN THE CONSTRUCTION SITE

CSTCBs are a factory-made product, transported and erected in a building complex, made active, integral and cooperative with the other structures of the building organism by means of an in-situ completion casting. CSTCBs are manufactured by means of an industrialised process using suitable plant and appropriately organised structures and techniques. The factory must be equipped with a production control system. The product quality management system must be set up in accordance with the applicable standards and certified by an independent, accredited third-party body.

The production and erection of these hybrid beams generally follows the subsequent scheme [23]. It starts with the plant production technical manager receiving the building design concept and operation instructions including the welding conditions. Received materials and parts are subject to acceptance inspections and non-CE-marked materials are taken in for sampling in accordance with relevant standards or specifications. The beams are fabricated with adequate, effective technology, personnel or operators, and established procedures. Process and product control, including those of the concrete, take place in a certified quality system according to UNI EN ISO 9000.

Transport documents are prepared, containing handling, assembly and installation instructions.

The transport is a crucial and important step: these prefabricated beams are delivered by trucks to the construction site. If the length of the elements is higher than 13,5 m, they are identified as abnormal loads and the road transport must be accompanied by safety cars. If the length surmounts 30 m (in general only for bridges) the abnormal load must be escorted by safety cars and local authorities. In the practical case study, the beams will be delivered by truck and eventually the street adjacent to the site will be closed to simplify the procedures of unloading and handling.

The beams and all the additional reinforcements are delivered to the site with all original documentation. The girders and any supplementary elements are assembled according to the specified instructions of the producer and between the tolerances specified. The main common procedures are:

- Demoulding the columns and levelling the support surface;
- Laying the prefabricated beams (Figure 153);
- Binding of the beams with the connection elements;
- Installation of the of the additional longitudinal reinforcements of the beams;
- Placing of the additional reinforcement of the slabs;
- Cast the filling concrete.

Site inspections are generally conducted to ensure correct installation and compliance with quality systems (where applicable). After the curing of the concrete casting, the temporary supports are demounted and the CSTC beams are put in service.

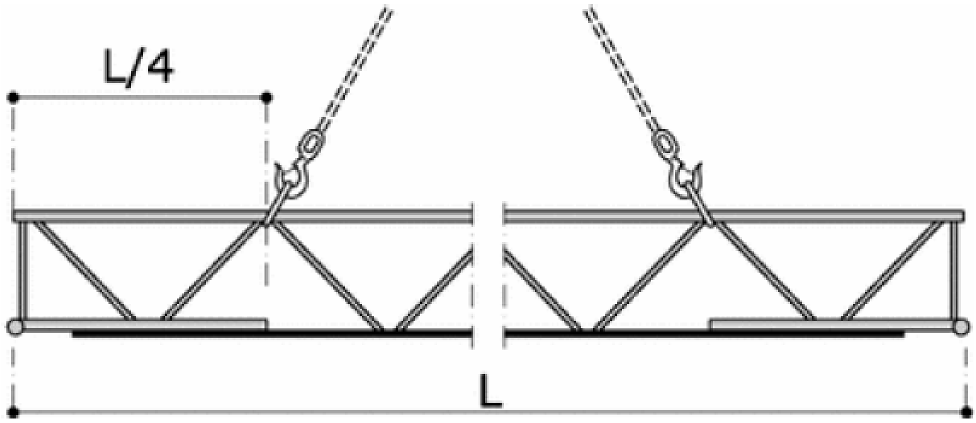

Figure 153 Particular of the handling of the truss element.

From a logistics perspective, the use of CSTCBs facilitate the ability to parallelize site works, with beam erection and slab casting progressing concurrently over sequential time periods without interference from other jobs. That advantage in sequencing lead to significant time savings and contribute to increase project efficiency. Moreover, less work has to be done on site and less formwork and props need to be used contributing to a safer working environment.

In conclusion, from the use of CSTCBs in the Ciriè construction site, their technical adaptability, simple installation and the compatibility of the materials used with modern construction processes are highlighted. These merits of CSTCBs combined with the structural performance validated in design verification, assess the potential of CSTCBs as being a very competitive alternative method of construction.

5. FINAL CONSIDERATIONS

5.1 CONCLUSIONS AND FUTURE DEVELOPMENTS

The thesis aimed to explore the vast field of precast structural components and investigate their potential application in modern construction, with a particular emphasis on Composite Steel Trussed Concrete Beams (CSTCBs). From a wide scenario of precast systems (Chapter 2), through the typological and material characterization of precast beams (Chapter 3), finally, to the successful real-world case study (Chapter 4), the research has proven that CSTCBs are technically feasible, suitable for building purposes, and efficient and effective when properly inserted in the building procedure.

Chapter 2 provides an extensive view of precast elements, ranging from the general characteristics to a detailed historical and technical examination. The historical overview follows the development of prefabrication though various eras, from the early developments of reinforced and prestressed concrete in the 1920s, to the period of mass production after World War II, the lightweight and long-span innovations of the late twentieth century, and the most recent tendencies linked to sustainability and automation. This evolution shows how precast production step by step integrate industrialisation, quality control and modularity to construction process. The dissertation then turned to the analysis of the various typologies of precast systems, distinguishing between building scale, component scale, and connector scale. At the building level, structural types including portal frames, skeletal frames, wall frames and cell systems as well as their functional types for residential, commercial and industrial functions have been investigated. At the component scale beams, columns, slabs, walls and stairs are addressed along with their structural functionality and manufacturing features. At the connection level, the various connection typologies (force-locking, cohesive and form-fitting) and their impacts on structural performance are presented. The chapter gives then special emphasis also to materials involved in the precast production, particularly into concrete mix design, reinforcement and fittings. Altogether, this chapter highlights how precast elements are not just a cast-in-place alternative, but an independent industrialized system, with the potential of fostering speed, safety and quality provision.

Chapter 3 focuses on the analysis of precast beams with specific reference to Composite Steel Trussed Concrete Beams (CSTCBs), which represent one of the most advanced and adaptable solutions within modern prefabrication. Initially, the chapter studies the classification of general types of precast concrete beams from very simple rectangular or L-shaped forms to more complicated ones such as I-beams and rafter beams, and shows how geometry and reinforcement types can be adapted to suit span length and loading requirements. It then focuses on CSTCBs, providing historical notes, a general description of their hybrid nature and a detailed classification of the different configurations. The structural response is investigated in depth, describing the two basic working stages. Such two-stage behaviour is one of the factors why CSTCBs are highly efficient, where no temporary supports are required and construction time is reduced. The chapter also discusses the long-term effects such as creep and the major design checks required by standards. These include checks for bending, shear and deflection, both in the temporary and permanent stages. The result from the analysis is that CSTCBs are able to use the best properties of both steel (light-weight and tensile strength) and concrete (compression strength and durability) and offer a competitive structural solution, particularly suitable for medium and large-scale projects where rapid construction and high performance are key requirements.

Chapter 4 applies the theoretical framework to a real-world construction project located in Cirié (TO). This serves as crucial mean to verify the practical implications of CSTCBs on an actual building site. The chapter starts with an examination of the building functional and structural requirements, then goes on with the development of the structural model, giving a detailed analysis on the loading conditions. A crucial point of the case study is the comparison between two structural solutions: conventional reinforced concrete (RC) beams and CSTCBs. Although widely utilized, the RC approach involved longer construction times, extensive formwork requirements and greater (and more complex) on-site labour. On the other hand, CSTCBs present notable advantages such as faster assembly due to prefabrication, elimination of temporary formwork, enhanced site safety and minimized interference with subsequent construction phases. Furthermore, the structural performance of CSTCBs in terms of strength, deflection control and overall stiffness meet satisfactory standards. The analysis also addressed practical considerations such as beam transportation, crane operations and alignment processes, emphasizing that although CSTCBs demand precision in manufacturing and installation, they significantly simplify the construction process.

One of the key contributions of this study, is the broad assessment of the structural performance of CSTCBs as both in their two stages of construction. In Phase I the steel truss works as a structural self-supporting element, minimizing external temporary formwork and facilitating site works. At Stage II, when the cast-in-place concrete is cured, the structure becomes a composite continuous beam having higher stiffness and load carrying capacity. This change in state has been assessed by means of analytical models, regulatory demonstrations and performance calculations that incorporated also the results of time dependant phenomena, in particular creep.

The case study of the building in Cirié has been used as a tool for the application of the theoretical concepts introduced in the previous chapters. The structural model has been made in DOLMEN, loads and limit states have been analysed. A significative CSTCB has been then chosen to make a comparison with the correspondent traditional RC beam solution under the same design conditions. The comparison assessed that CSTCBs not only meet structural requirements, but also offer significant benefits in terms of construction speed, reduced on-site labour, and improved installation logistics. Furthermore, the on-site application highlighted the adaptability of CSTCBs to real construction restrictions.

The combined results obtained confirm that CSTCBs is a robust and high-quality solution for medium to long span structural applications, in particular where time, modularity and site effectiveness are a priority.

Despite these benefits a broader adoption is currently limited by the presence of some open issues that will be crucial in future developments [25]:

- A very limited number of studies are currently available on topics as the investigation of long-term performance and the analysis of geometrical and size effects. Further full-scale experimental tests and improvements in digital tools and parametric model are needed to analyse in depth these aspects.

- Standardised rules and calculation models are not yet implemented in the currently available building codes, thus providing an important lack of standardisation. Additional efforts will be required in the next years in order to achieve this fundamental objective.

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