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Unshoring Process of a Temporary Pillar, in a Seventeen-Storey Building in Sant Adrià del Besós

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Abstract: In the new construction of a seventeen-storey building, a provisional prop of fourteen-meter height, horizontally braced on two intermediate levels, has been used. Despite the fact that structural logic suggests that it can be cut without having any added safety precautions, the structure of the building, made up of cores and reinforced concrete slabs working spatially, indicates that certain mechanisms be introduced so that, in the event that different and worse behaviors than expected are detected, the process can be stopped and the consequences of the new situation observed can be analyzed. For this purpose, two pairs of four metallic cantilevers were introduced at mid-height with four hydraulic pistons. In addition, the best position of a series of strain gauges as well as transducers were analyzed. At first, a load test was carried out to check that the brackets worked correctly. Once this step was verified, the abutment was cut, and the results were read. The results of stresses and deformations were compared with those expected, always being satisfactory. Finally, four provisional profiles were placed in case after a few days the structure suddenly gave way. The research focuses on obtaining an efficient control system and achieving total security throughout the process, with the comparison of the results strictly necessary for this case. Few resources were used so as not to make the work excessively expensive. We have found important divergences, on the side of security, between calculation and reality. We have also considered that the construction process has an impact on the final results. In the same way, the rigidity of the temporal abutment must be considered before the calculation. All these factors have generated a lower-than-expected deformation in an 8 m cantilever.

Keywords: unshoring; high-rise building; hydraulic pistons; temporary prop; monitoring; robustness; rigidity; auscultation

Citation: Costales Calvo, I.; Font i Basté, J.; Gimferrer i Vilaplana, X.; Llorens Solivera, M. Unshoring Process of a Temporary Pillar, in a Seventeen-Storey Building in Sant Adrià del Besós. *Buildings* **2024**, *14*, 3436. https://doi.org/10.3390/buildings14113436

Academic Editors: Shaohong Cheng, Harry Far and Sathees Nava

Received: 18 July 2024
Revised: 15 October 2024
Accepted: 24 October 2024
Published: 29 October 2024

1. Introduction

Specific structural health monitoring systems (SHM) [1–4] required for the implementation of a new building structure are usually left to the discretion of the designer [5–7], unlike what happens in civil engineering, where it is a very frequently used tool. Before the drafting of the first Eurocodes back in 1975, the description of these systems, as well as the way of applying them to building works, was very scarce.

Fortunately, as with other quality control systems for materials [8,9], it has been incorporated in a generalist way. Several conditions are decisive: (1) the lack of need for complex control monitoring in most building works; (2) the difficulty involved in establishing the obligation of these mechanisms, as well as the determination of minimum controls; (3) the difficulty of reversing a traditional way of working for many years without the need for excessive control; and (4) the cost, which despite being able to be solved with a reasonable investment, is frequently used to improve the final quality of the construction systems.

Project audit systems [10], such as those carried out internally by some construction companies [11], or those carried out by some insurance companies [12], favor the good result of the implementation of the structures. In addition, the preparation of shop drawings by third parties can serve to reinforce the review. And finally the monitoring carried out by the Construction Management, which is not always part of the definition of the executive project.

No additional monitoring measure [13] is usually necessary to remove a temporary prop used to build a structure. The reason is none other than understanding that the actions applied during the construction phase are less than those defined in the project, and that the safety coefficients [14,15] during the construction phase are more adjusted than the final ones. When we say that monitoring is not carried out, we understand that, if visual and effective topographic monitoring is carried out [16], and that, despite the above, undesirable behaviors may occur that can be corrected or reinforced, such as deformations greater than those expected, or slight cracks.

Likewise, the current calculation methodology, despite having been improved with the use of personal computers that are faster every year, as well as the calculation systems [17], does not allow a calculation process to be carried out in phases [18]. This is (1) because they are incompatible with the time required for the definition of the project, (2) because the cost of these programs, as well as the labor, is considerable, (3) and because not considering the phases is not usually a reason for later pathologies, except in rare cases.

The hypothesis of considering a temporary strut is not usually analyzed in detail in the project phase, or at least, the structure is not usually checked under this hypothesis. A forecast of the material is made, as well as its section, but it is not always usual to consider the influence of its rigidity [19,20]. The calculation to be carried out is usually simple: it is the case of a bar subjected to simple compression, considering a small eccentricity of the load, and considering the global imperfections of the bar, to avoid the second order calculation [21].

In this research work, we have achieved a simple, reliable, and reasonably economical method to have a safe process at each stage. We have found important divergences between the calculation made and the final real behavior, fortunately they were always considered in the project with a safety coefficient greater than necessary. We have also considered that the construction process has an impact on the final results. In the same way, the rigidity of the temporal abutment must be considered before the calculation. All these factors have generated a lower-than-expected deformation in an 8 m cantilever.

2. Brief Description of the Building

The building called "Tembo Barcelona" is located at Ramon Llull, 479 Street, in Sant Adrià del Besòs, in an area where the housing built during the mid-twentieth century was designed for the working class [22]. The building's typology consists of three floors below ground level, a ground floor, and seventeen floors. It is considered an EGA high-rise building [23] under local regulations. Each typical floor has a built area of approximately 1300 m^2. The structure was designed mainly using reinforced concrete slabs, with the exception of some metallic or mixed (steel-concrete) elements [24]. The building offers a long cantilever above the main entrance, formed by 14 floors. To support this cantilever, a triangulation was introduced in the façade, which is partially hidden behind the outermost skin of the terraces. The shoring system was conceived in agreement with the architect author of the building, Jaume Font i Basté (D388 Arquitectura), foreseeing two diagonals: one approximately 36 m long, and the other almost 15 m in the opposite direction (A and B, Figure 1), which circumvent the windows of the building. It was not possible to introduce downstand beams on each floor, nor the consideration of active forces [25]. The construction company that was awarded the work is called Copcisa. SCS-Cullaré

Sala Structures completed the metal structure, which was initially calculated by BEST Costales Jaén.

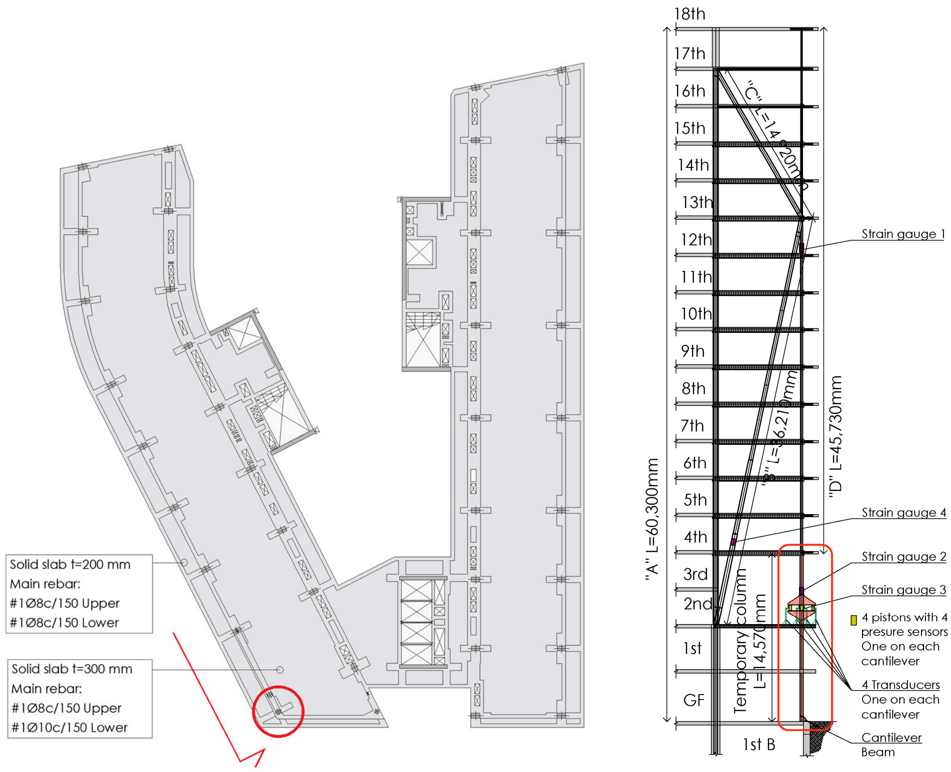


Figure 1. The image on the left shows a typical floor view of the structure. In the lower left, the provisional pillar is located in a red circle. In the image on the right, a part of the elevation of the frame that contains the shoring is shown. In the double height formed by the second and third floors, the provisional pillar can be seen, which has continuity on the ground and the first floors.

In order to build this cantilever, a temporary pillar of 14 m height is proposed, which will be removed when the structure is completed in its entirety. A key factor in the design of the shoring system was the rigidity of the structure in all its stages [19,20]. The computer analysis was key for the calculation [26] and monitoring of the work [27]. However, during the shoring process and subsequent removal of the prop, it was necessary to exhaustively monitor the path of the stresses and deformations of the elements involved.

3. Description of the Structure of the Building

The first impression when you analyze the solution used is that other triangulations are necessary towards the opposite part of the cantilever, acting as a counterbalance of the system, in the same plane towards the back of the pillar that supports the diagonals of the cantilever. Finally, no other diagonals are necessary apart from the ones described. The consideration of a spatial work of the slabs will be reasoned later to solve the problem.

The V-shaped floor plan is grid by pillar distance of approximately 7.2 m and up to 8 m in some specific locations. A single span and two cantilevers, one on each side, which reduce the positive moment in the center of the span, form the typical cross-section of the floor. Slender concrete pillars were designed, taking advantage of the fact that the concrete cores stiffen the building. The horizontal structure was designed using solid reinforced concrete slabs for several reasons: (1) the fire stability required by firefighters was 120 min; (2) a solid slab requires a lower thickness compared to other light options, so the free height of each floor was slightly greater, favoring the placement of the facilities; (3) and to a lesser extent, to improve the acoustic behavior of the building between floors. Hydrostatic thrust, as well as the general excavation, is also reduced by using thinner slabs. Three concrete cores, placed where the elevators and stairs are located, brace the building horizontally. Seismic forces were not taken into account in the design phase of the building, as it is not mandatory according to current regulations NCSE-02 [28]. The high exposure to wind [29] suggests that the earthquake effect would not be a more unfavorable hypothesis than the wind hypothesis considered in the calculation.

The pillar that does not have continuity is indicated in Figure 1 with a red circle; it is located in a corner of the building's floor plan. In order to complete the construction of this area, a temporary pillar was positioned. Formed with two HEM-280 steel sections, the pillar is fourteen meters high. The profiles were joined together by welding their flanges, forming a box that increased its buckling resistance. This pillar, which was the equivalent length of four stories high, rested on the end of a slab near the crowning girder of the slurry wall. An external provisional metal cantilever beam was proposed at the foot of the metallic pillar, which would be removed when the unshoring process was finished (see Figure 1 "Cantilever Beam" in the bottom right). Meanwhile, the pillar was horizontally fixed by the ground-floor ceiling and the first-floor ceiling, to work with six meters of buckling length. To fix each level, four vertical pieces of neoprene were placed on the perimeter of the hole left in each slab, maintaining contact between the slab and pillar. The temporary prop remained in service during the construction phase, until the seventeenth floor was reached. Once the main structure was completely finished, it was time to remove the prop.

The tributary area of a typical pillar is estimated at 30 m^2, so the axial strength of each floor is evaluated at 530 kN. This force is not real because the diagonals cross the slabs, giving them support and therefore reducing the load. This calculation is made to know that the mobilized force reaches 8900 kN in ELU, considering in the sum the weight of the façade of that area.

The maximum axial compression supported by the provisional strut is justified below:

A_s = 48,040 mm^2; $I_{yy'}$ = 194,516 × 10^4 mm^4; $I_{zz'}$ = 125,935 × 10^4 mm^4; Classe = 1; $\alpha_{yy'}$ = $\alpha_{zz'}$ = 0.49; $N_{cr,yy'}$ = 111,988 kN; $N_{cr,zz'}$ = 72,505 kN; $\lambda_{yy'}$ = 0.34; $\lambda_{zz'}$ = 0.43; $\phi_{yy'}$ = 0.59; $\phi_{zz'}$ = 0.65; $\chi_{yy'}$ = 0.93; $\chi_{zz'}$ = 0.88; $N_{b,Rd} = \chi_{min} \cdot A_s \cdot f_{yd}$ = 0.88 × 48,040 mm^2 × 275/1.05 N/mm^2 = 11,110 kN

During the definition of the executive project, it was determined that the strut would be eliminated with oxycutting [13] using a diagonal cut method, without taking any additional precautions. As an added precautionary measure, deformation would be checked by topography or by a precision gauge strongly fixed to the ground. An inclined cut is usually executed to remove the temporary prop. If a horizontal cut were made, it would not work, because it has been proven that, once the pillar is cut, both ends are rejoined due to the deformation of the system. While the inclined cut causes both parts to slide on the cutting plane, automatically obtaining the separation of the system.

It is common not to take any precautions when a prop is shored, since it must be considered that the calculation first, and the execution later, have been sufficiently checked. Furthermore, at the time of removing the prop, the structure is loaded with practically half of the actions accounted for in the calculation: permanent and live loads are practically non-existent. In addition, the regulatory safety coefficient during the construction phase is lower [14,15,30]. As it is a very delimited area, the live loads can be eliminated, preventing access on each floor.

However, contrary to what has been explained so far, during the construction phase, it was considered that if the final deformation observed in the system, once the undercutting had been carried out, was considerably greater than calculated, there would be no way to recover that deformation or, at least, to stop it. The decision was not made for reasons of calculation reliability, but under the consideration that the shoring worked as a spatial system, and could eventually have a different response to that analyzed in the calculation model. In this case, the construction process could generate a final behavior different at the expected one. It is probably fair to say that the total cost of the operation, which amounted to approximately twelve thousand euros, was well invested in order to have a clear document in the future that would rule out later problems.

When it is said that it is a spatial system, it means that the building does not have any other diagonal than those shown in the portico, the moment of overturning being retained by the slabs, which in turn are embedded horizontally in the core. It is this core that prevents the building from tipping in the direction of the overhang. It is also responsible for transmitting up to 80% of the wind actions to the foundation.

On some floors, these leaning pillars will be partially visible in some rooms (Figure 2), right on the façade plane, between the living room and the exit to the terrace. In 1966, at the John Hancock Center Tower in Chicago (Fazlur Rahman Khan, Architect, Skidmore, Owings and Merrill, Structure), there was a concern about what would happen to these rooms that incorporated diagonals in their facades during the calculation and design process. It was one of the first high-rise buildings to experiment with a solution with exterior bracing, a solution that would later be used by other buildings, as in the case of the Hotel Arts in Barcelona, the work of the same authors. The end result was that not only were there no problems in their demand, but they were also the most sought-after spaces, as these diagonals incorporated a formal and spatial quality to the interior, different from that of any apartment in Chicago [31].

Figure 2. Image of the hotel structure completed in January 2022. You can see the temporary vertical pillar (indicated with a red rectangle), which has not yet been removed.

4. Description of the Frame That Contains the Shoring

The frame that completes the shoring is formed by the following:

1. A composite steel-concrete vertical pillar (A), which goes from the foundation to the roof. It was designed as a composite, in order to facilitate the union with the rest of the metallic diagonals, which will be defined below;

2. An inclined steel diagonal (B), HEM 400/320 with added plates, goes from the second floor to the thirteenth floor. It is 36.2 m long and 78° inclined. In the foot, it is born from the pillar described above (A), and in the head, it coincides with the beginning of the vertical metallic tension bar (D) and with the end of the top diagonal (C) that will be described below;
3. An inclined steel diagonal (C), HEM 320 with added plates, goes from the head of the previous diagonal to the ceiling of the sixteenth floor. It is 15 m long and 60° inclined;
4. A vertical steel tension bar (D), HEM-200 with added plates, that completes the frame, goes from the third floor to the twelfth floor ceiling. Above this floor, it continues as a compressed pillar. As a particular characteristic, it is worth mentioning that this element will be compressed during the construction phase, and will be subjected to traction in its final state.

The entire vertical of the brace (D) was built with a slight counter-deflection so that at the time of de-shoring, that point did not have excessive deformation. The final axial forces that were calculated during the definition of the executive project, both in the work process and in the final phase, are detailed below in Table 1.

Table 1. Final axial forces considered in the frame.

Element (See Each Position in Figure 1)	Axil Force with the Prop. Upper Floor (SLS)	Axil Force with the Prop. Lower Floor (SLS)	Final Axial Force Upper Floor (SLS)	Final Axial Force Lower Floor (SLS)
(A)	−159 kN	9932 kN	14,000 kN	18,500 kN
(B)	2470 kN	4252 kN	2300 kN	8050 kN
(C)	1605 kN	3003 kN	−100 kN	−690 kN
(D)	2204 kN	6800 kN	−510 kN	−186 kN

During the design process, it was considered that the final axial force transmitted by the provisional prop could be evaluated at about 3000 and 6800 kN in SLS (Serviceability Limit State). The reason why there was such a high variation between the two results corresponds to the fact of whether or not the construction process was considered. Finally, the most unfavorable case would be taken, in anticipation of possible delays when carrying out the shoring.

5. Description of the Structural Solution Adopted for the Unshoring Process

During the construction phase, the decision was made to progressively remove shoring, placing two pairs of four cantilever beams (Figure 3) in the two main directions, and on two levels separated by about 575 mm [1–4]. This distance was determined by adding the maximum deformation observed in the general calculation models, the height of the jack necessary for the required force, and finally, adding a margin in accordance with the maximum acceptable stroke. These types of techniques are commonly used in many buildings, as is the case with many historic ones [32–34].

The control equipment [35] proposed by the company 3Stech Smart structural Sensing technologies, to monitor the process, is detailed below:

- Four Hydraulic pistons of 3.000 kN capacity each [36–38]. They were placed in a cross-shape form, one on each arm of each support. They had a maximum stroke of 150 mm and a maximum opening of 575 mm;
- Four strain gauges (S.G): these are sensors that determine the axial load of the inspected elements, depending on the elongation observed. They have been placed according to Figure 1: in the head of the tension element, which is an extension of the provisional pillar (S.G. nº1); in the web of the temporary pillar (S.G. nº2); between the cantilever beams (S.G. nº3); and in the inclined pillar (S.G. nº4);

- Four displacement transducers: they control the vertical movement of the analyzed points, in this case, the tip of the metallic cantilever beams, getting to know at all times of the process the inclination of the plane formed by the four beams;
- Four pressure sensors: they give information on the pressure of each piston, which can be corrected at any time; one on each metallic cantilever beam.

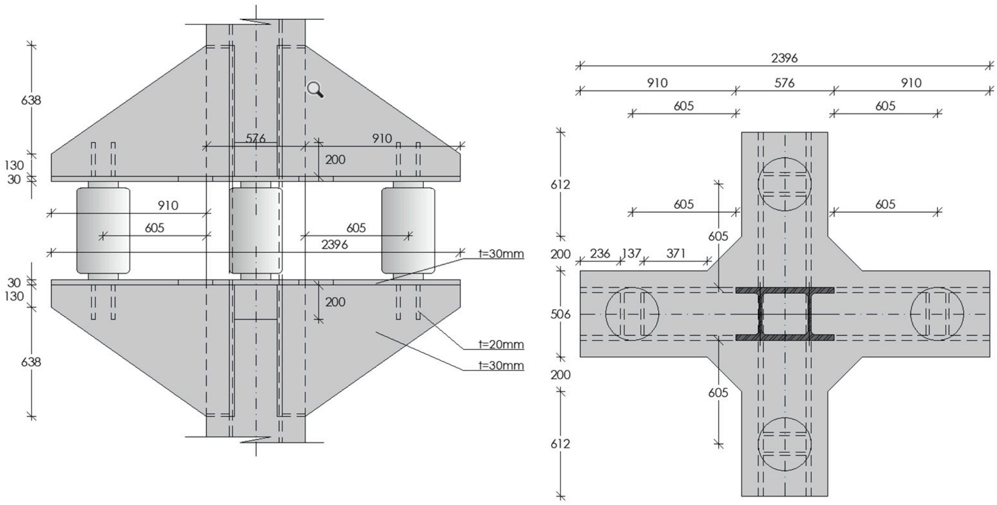


Figure 3. Proposal for the cantilever beams, made by GMK Associates and 3Stech.

All the information was recorded on a device connected to the sensors described above, which gave information at all times during the intervention. The verification of the calculus of the cantilevered beams was carried out manually, as shown below, using the method "cantilever beams with unstiffened web", which was described by Dr. Francisco Quintero in the UNEDguides [39], and which was subsequently included in the Spanish steel regulations EAE [40] in its article 61.5 "Support on unstiffened bearing".

C = 767 mm; d = 605 mm; t = 30 mm; λ = 0.587; CE = 1.721; $M_{pl,Rd}$ = 752,433,000 N·mm; V_{Rd2} = 3170 kN

As each bracket has 2 plates in its web [41–44], it can withstand a shear of approximately 6340 kN. Considering the four corbels, 25,360 kN would be reached. Apparently, it can be understood that this is an excessively high figure. Later, it will be understood that this oversizing was a precaution for the process.

The final deformation was estimated between 20 and 30 mm. It was important that the maximum expected deformation was defined in a loose way, since the piston had to be able to allow this stroke with sufficient clearance: if the piston had a maximum allowable stroke less than expected, it could not be removed and the process would have failed. This deformation was difficult to calculate because the construction process of the concrete structure took about six months to achieve, and while some pillars were concrete, others were composed of metallic concrete and had a different time-dependent deflection. Therefore, the instantaneous deformation depending on the real Young's modulus, the construction process, and the shrinkage and creep of the material will influence the final result.

In order to be able to contrast the results that were considered during the work by using formulas, a finite element model was generated under a university license, several months after the end of the work. Therefore, it is a verification made a posteriori for the preparation of this article. It was carried out with the ANSYS program [17,45], which considers the buckling of the plates in second order. The maximum stresses obtained at full load are generally less than the yield strength of the material f_y = 355 N/mm^2, as shown in Figure 4. At a very specific point, tension peaks of around 506 N/mm^2 are observed, but their location allows the integration of stresses in the area. This analysis gives a result lower than the maximum allowed stress. The results obtained are in any case in accordance with those expected during the construction phase.

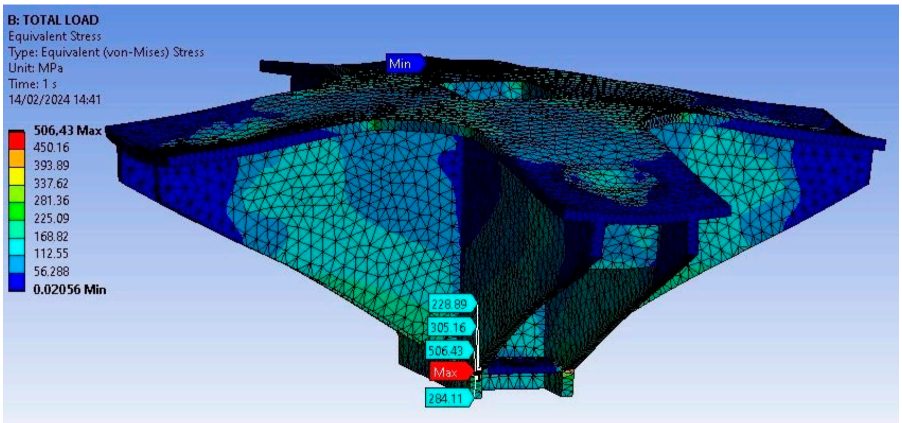


Figure 4. Analytical model of the cantilever beams. Maximum tensions in the Ultimate limit state (ULS).

6. Description of the Unshoring Process

During the previous days of the implementation of the shoring process, the corresponding tests of the welds and the designed metal elements were carried out. The result was positive in all cases. The next step was to ensure that the temporary pillar was firmly welded to the first supported floor: if the pillar was cut, it was important that the upper section did not fall on the workers.

On 14, 15, and 16 March 2022, the process of pre-loading the cantilever beams was carried out, as was the subsequent de-shoring process.

- Pre-loading process and subsequent unloading of the cantilever beams: On 14 March 2022, all control elements were placed and checked. The system was then placed under load using four steps of equal magnitude, with a final load of 6800 kN, which represents 85% of the load in ELU. Once this force was reached, it was unloaded without having started any cutting process. This previous step was, so to speak, the "insurance" of the "insurance", that is, a load test of the system. The proposal can also be understood as a prestressing of a metal structure for its control [46]. The final deformations obtained in each cantilever beam are detailed below in Table 2:

Table 2. Displacements obtained in the cantilever beams during the pre-loading process.

Stage	Transferred Load (kN)	Load Obtained in the Pillar (kN)	Displacement of the Cantilevers (mm)			
			TR 1	**TR 2**	**TR 3 ***	**TR 4**
1	1700	1950	0.39	0.52	0.62	0.41
2	3400	3982	0.81	1.03	1.26	0.83
3	5100	6067	1.23	1.58	1.92	1.24
4	6800	8177	1.67	2.15	2.57	1.68
Leftover	-	-	0.08	0.16	0.04	0.10

* Transducer 3 takes relative readings of the displacements between the upper and lower cantilever beams.

During this process, it was possible to verify a slight warping, barely insignificant, of one of the plates of one cantilever beam. No image is attached since it was barely noticeable. This warping occurred when one of the sheets was subjected to a greater compression than was finally necessary, possibly because the piston was not perfectly centered on the beam and one of the two sheets was more requested than expected.

In this type of joint, it is always interesting to introduce a small stiffener at the beginning and end of the outside of the diagonal edge, to improve the embedding of the core with the base plate and with the pillar. Despite the fact that the theoretical model worked and that the matrix calculations in the second order did not represent stability problems, this slight deformation was appreciated and was not a cause for concern, nor for subsequent repair, because the force introduced was greater than calculated, expected, and finally obtained. Therefore, this negligible unexpected deformation made the loading process a good practice.

Finally, the load introduced into the system was removed, which "self-compensated" without altering the system. It is necessary to say that the prop was checked and had enough margin to be able to receive the prestress load plus the one introduced in a very localized area, between the metallic cantilever beams.

- Pillar cutting process: On the following day, 15 March 2022, after placing and checking all the control elements, as well as the pistons, a prior force of 900 kN was introduced to the cantilever beams, to pre-load the pistons. This action represented approximately 13% of the final estimated load. Once the entire system had been checked, the space remaining between the two groups of four brackets was cut with oxyfuel. The pillar-cutting process was carried out at a height of 1.2 m above the first-floor ceiling, that is, in the lower part of the double height, making two flat cuts about 150 mm apart from each other. In that position, the oxyfuel team could work comfortably. First, an outer flange was completely removed from an HEM section. Then, the opposite flange of the other HEM section, to finish cutting the four flanges. In this way, this section was weakened, allowing it to rotate itself. Finally, the central core was removed, completely freeing the pillar, and progressively transferring the action to the pistons.

The load that these pistons finally transmitted was 2825 kN, slightly less than half of what was expected. On each floor, a live load was foreseen, as well as a dead load, which in the end was only applied to some of the lower floors. The construction process was also decisive in this reduction, since the structure was loaded progressively, and not completely, as usually happens in a matrix calculation model.

During the cutting process, no load increases or significant movements were detected until approximately 4800 s (80 min) of the process was reached. Movements of approximately 2 mm were observed.

After this point and until 6500 s (108 min) of process, the movement stabilized at the previously defined figure, between −16.8 mm and −19.65 mm (Figure 5). This deformation is insignificant for the final lowered load. It also gave us an orientation of the final deformation of the shoring when the building was fully loaded.

At the end of the work of cutting the provisional pillar, and after a reasonable amount of time had passed, the load was progressively removed from the pistons, observing a final deformation of the structure of 18 mm, which was less than the maximum expected in the worst case, 30 mm. The pistons were left about 10 mm from the structure for 48 h, in anticipation of possible settlement adjustments. There was no significant increase in deformation, although there was a slight variation in the distribution of stresses in the structure, as can be seen in the following Table 3.

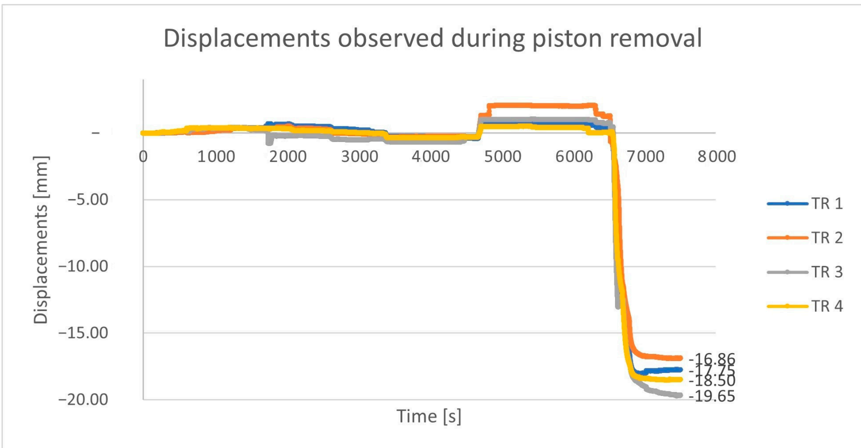


Figure 5. Deformations were observed in the four pistons during the cutting process. The results were recorded by transducers, one placed in each piston.

Table 3. Final loads and displacements.

	Loads (kN)				Displacement (mm)			
STAGE	**Piston Sum**	**Cut Pillar**	**Pillar 12th Level**	**Inclined Pillar**	**TR 1**	**TR 2**	**TR 3**	**TR 4**
Start cutting	900	0	0	0	0	0	0	0
Final cut	2748	−160	−25	48	0.81	2.07	1.0	0.5
Piston-lowering	0	2825	925	−1458	−17.75	−16.86	−19.65	−18.5
After 48 h	-	2819	1036	−1305	-	-	-	-

7. Results and Discussion: Follow-Up of the Deformation in the Following Days

During the next day, as a safety measure, the pistons were not removed as shown in Figure 6. This decision was considered so that if the structure were to settle suddenly, subsequent corrective action could be taken, leaving the structure momentarily retained. Since the hydraulic pistons had to be used urgently on another job, and since renting more days would significantly increase the final cost of the operation, the pistons were replaced with pieces of HEB sections, leaving a separation of approximately ten millimeters from the upper cantilever beams. Thus, if the system went down due to some unconsidered problem, the piece remained stabilized without the possibility of falling.

The deformations observed during the following month were variable, but in no way understood as a process without end. The changes that were collected in the reading, and can be found in Figure 7, were attributed to several factors:

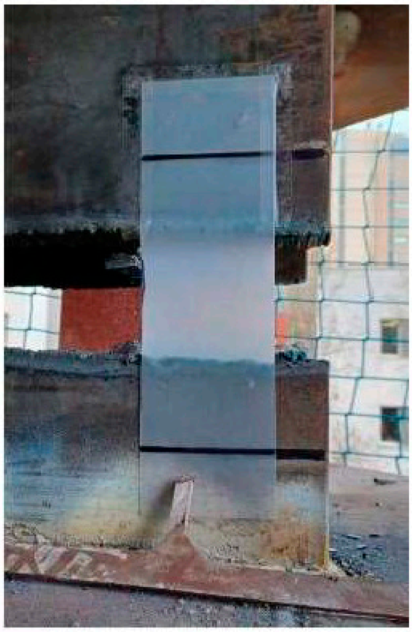
- Time difference: it seemed that solar radiation might have affected some cases.
- Disposal of materials in various levels: during the days after the cut, some partition walls were built that increased the weight, and therefore the deformation of the cantilever area.
- Small inaccuracies in the measurement: although the measurement system was at all times careful and efficient, as it was not an electronic lecture system, it could contain slight deviations. It was not understood, in any case, that the results shown were incorrect. The method used to track the deformations consisted of the use of a transparent millimeter scale and the measurement of two indelible marks placed at the two ends of the cut temporal pillar.

Figure 6. Image of the system after the pillar is cut and separated from the system. In the top head of each piston, a space is visible. The transducers were removed before this photo was taken.

TEMBO APARTMENTS

Deformation control of the singular pillar

DATA	TIME	DEFORMA-TION
March 15th	13:00 h	Begin/Cut
March 16th	9:00 h	0.0 mm
March 17th	9:00 h	0.0 mm
March 18th	10:00 h	1.0 mm
March 21st	8:30 h	0.0 mm
March 22nd	9:00 h	0.0 mm
March 23rd	9:00 h	0.0 mm
March 24th	9:00 h	1.0 mm
March 25th	8:30 h	1.0 mm
March 28th	9:00 h	1.0 mm
March 29th	9:00 h	
March 30th	18:00 h	2.0 mm
March 31st	9:00 h	1.0 mm
April 1st	8:30 h	1.0 mm
April 4th		
April 5th		
April 6th	16:30 h	4.0 mm
April 7th	9:00 h	2.0 mm
April 8th		
April 11th	9:00 h	2.0 mm
April 12th	9:00 h	2.0 mm
April 13th		
April 14th		
April 15th		
April 20th	10:30 h	3.0 mm
April 21st	16:00 h	3.0 mm
April 22nd	9:00 h	
April 25th	9:00 h	3.0 mm
April 25th	12:30 h	4.0 mm



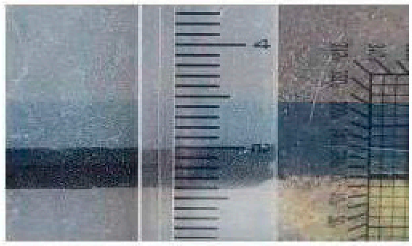


Figure 7. Manual monitoring of the deformations made by David Sanz Engineer, head of technical office of COPCISA, during the following month after the process of the cutting process of the provisional pillar, has been completed.

8. Conclusions

In this document, a solution designed to eliminate a provisional metallic prop (Figure 8) fourteen meters high, subjected to an action of 2825 kN in service (SLS), has been described. This process was proposed to have control over the final results of tensions and displacement. It was thus possible to ensure that the spatial system that makes up the structure worked as designed in the executive project phase. In order to have better control, the following registers were carried out and have been justified throughout this article:

- Axial forces in the truss: Table 1 details the stresses that were initially considered in the executive project for each element that makes up the truss, both in the case of the shoring structure and in the case after the removal of the provisional pillar. The results obtained in the structure, once the prop has been removed, can be found in Table 3. Comparison of these stresses indicates that the maximum load transmitted by the cats was 2825 kN, while a compression of 6000 kN had been applied. The difference in these figures suggests that the construction system, as well as the supposed overloads of use, was considered on the safety side. The construction process was not complete until the conclusion of the truss, so the construction floor by floor, as well as the rigidity of the provisional prop, was decisive. A phase-by-phase calculation was not carried out because it was excessively complex to analyze. Finally, it was decided that all the upper floors would be free of any other load than their own weight, and some of the facades that were planned to be built in this area were not executed. It is clear that the final forecast of the loads must be carried out on the safety side in order to know from the beginning the model of the hydraulic jacks used;
- Deformations observed in the four pistons during the cutting process: The deformations observed during the preloading process of the cantilevers, shown in Table 2, reached a value of up to 2.15 mm in one of the arms, and left an imperceptible initial deformation residue of up to 0.16 mm. The initially expected deformation according to computer calculations was 5 mm, so the results once again favored safety. It was not a minor action, since the beams were designed for 6800 kN previously introduced by the hydraulic jacks. The final deformations, which can be seen in Figure 5, were approximately 18.50 mm. These deformations refer not to the descent of the beams, but to that of the upper cantilever. They were also lower than expected, in this case 30 mm, for reasons parallel to those explained in the previous points; the beams
- Tensions in the brackets: The stresses and the final deformations in the cantilevers were checked manually using material strength formulas [22]. In this article we use a three-dimensional model under academic license (Figure 4) that, two years later, corroborated the initial calculations of the project, always remaining below the admissible steel resistance. The deformations achieved in the cantilevers also indicate the good response of the system;
- The subsequent manual monitoring of the deformations: At both ends of the cut abutment, two indelible and fixed marks (Figure 7) were placed to measure the deformations that occurred in the days following the cut. For a month and a half, a maximum deformation of 3 mm was accumulated caused by the gradual introduction of new loads in the building. The deformations varied throughout the day, possibly because the thermal expansions had a slight impact on the final result. The deformations were accepted because they were always in the expected range of less than 30 mm;
- Final monitoring of the transfer of the forces: As can be seen in the final image (Figure 9), the forces were transferred first during the cutting process of the column, then during the waiting or observation time corresponding to approximately 30 min, and finally during the unloading of the hydraulic jacks.

a. Values corresponding to the cut provisional abutment: To cut the pillar, an initial load of 900 kN is introduced by the hydraulic pistons. As the section of the pillar is reduced—progressively removing the flanges and then the webs from the section—the load is transferred to the pistons. Figure 9 shows the evolution of the loads of the different columns monitored in the cutting of the column. It is observed that before the discharge of the pistons, the value of the load supported by the pistons, 2748 kN, is practically identical to that corresponding to that measured in the progressive discharge of the pillar 2825 kN. The difference between the two values is approximately 3%. This similarity between both values shows that the maneuver was being monitored correctly. The resulting load is much lower than the estimated expected 6000 kN;

b. Values corresponding to the upper sections of the pillar (pillar corresponding to the 12th floor). It is observed how the pillar is unloaded. This must be interpreted as the axial changing sign, since initially it was compressed, and after the cut, it is tensile;

c. Values corresponding to the diagonal pillar: It is verified that the inclined pillar gains load. As can be seen in Figure 3, the displacements—as has happened with the loads—are markedly lower than expected: they are around 18 mm, with the expected values being in the order of 30 mm;

d. After 48 h of cutting the pillar, the variation in loads and displacements is checked with respect to the values measured at the end of the load transfer operation.

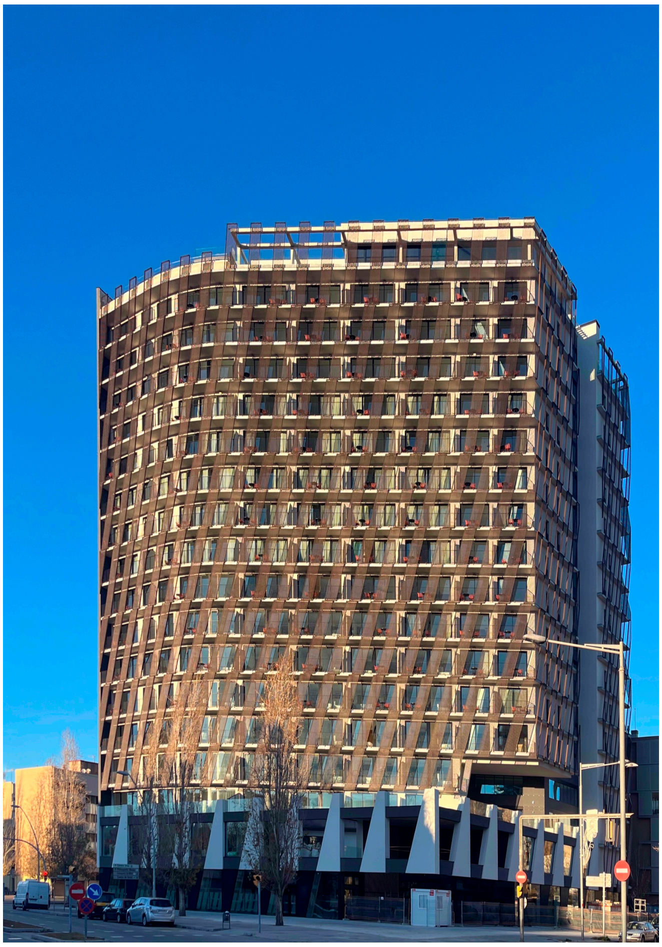


Figure 8. Image of the hotel finished in December 2023 without the provisional prop. It can be seen that the emptying of the corner favors the resulting volume making it lighter. The lower diagonal of the principal frame is difficult to locate.

Two factors have been key in this process: the rigidity of the provisional prop which, being 14 m high and having a bracket at its foot, has been deformed very slightly as the building was loaded, and on the other hand having had a triangulated structure that was not completed until the level of the 17th floor was reached. This process produced axial stresses lower in the temporal pillar than expected, and greater in the main pillar. Likewise, deformations smaller than those calculated were also collected. Despite the above, the structure was safe at all times.

The initial decision not to have any safety system is usually reasonable in almost all construction sites. Personally In other buildings in which we have intervened, such as the Forum 2004 building, designed by the architects Herzog and de Meuron, other control systems were proposed and finally rejected [47]. If a structure is designed with safety coefficients considering up to 40% more total load and is built with little more than its self-weight, the design, revision, or construction errors must be very significant for a collapse to happen. Initially, we were reluctant to accept this control system. However, for this specific case, where the torsional behavior of the building, and the due to he existence of steel elements and steel-concrete composites, was difficult to assess, our final opinion—taking into account that the final cost was easily assumed by the owner—is that it was a ggod investment to improve the final security of the works. The final appearance of the building was surprising, since the upper volume flies discreetly over the podium, with the diagonals hidden behind the outer skin (Figure 8).

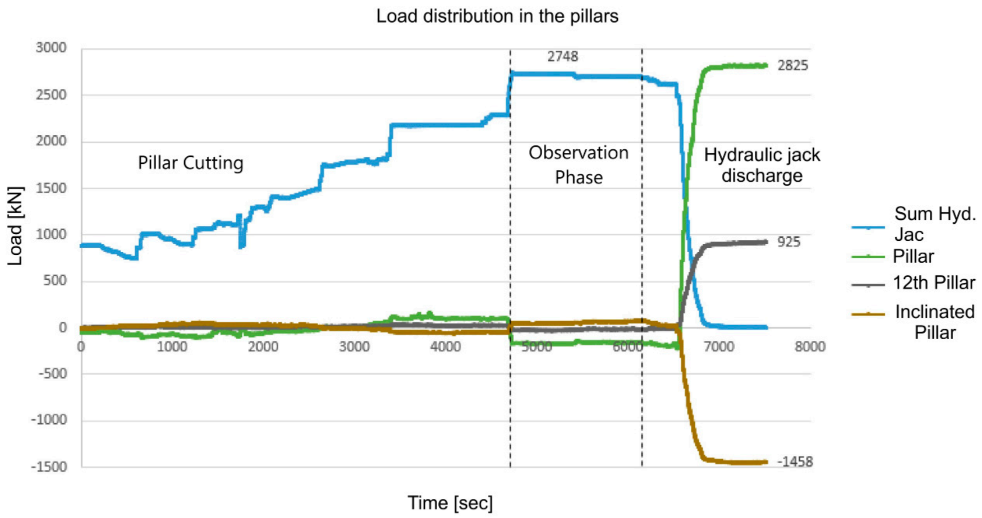


Figure 9. Load distribution in the pillars during the process.

Author Contributions: Software, I.C.C. and M.L.S.; Validation, X.G.i.V.; Formal analysis, M.L.S. and I.C.C.; Resources, I.C.C.; Writing—original draft, I.C.C. and J.F.i.B.; Writing—review & editing, I.C.C. and J.F.i.B.; Supervision, X.G.i.V. All authors have read and agreed to the published version of the manuscript.

Funding: This research received no external funding. This publication was funded by the Polytechnic University of Catalunya UPC.

Data Availability Statement: Data are contained within the article.

Conflicts of Interest: The authors declare no conflicts of interest.

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