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STRUCTURAL REINFORCEMENT IN METALLIC COVER DAMAGED BY CORROSION

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ABSTRACT: The steel structure buildings have shown strong growth in Brazil, especially in the execution of works with large free spans such as warehouses, gymnasiums and sports courts. However, with the progress of the execution of these projects it has been evidenced a demand for pathological manifestations, especially in the structure's joints, being corrosion the main pathology identified. Thus, this work aimed to conduct a case study on the Sports Gymnasium belonging to the IFTO - Federal Institute of Education, Science and Technology of Tocantins, in the city of Pedro Afonso in Tocantins, Brazil. It was built in steel structure with arch trusses. It was deactivated due to the work not being concluded, and also because the roofing system was damaged. By means of an on-site visit, corrosive manifestations were identified on the pieces, as well as disparities between the structure executed on site and the project proposed for the sports center. After mapping the roof, modeling the structure using software, and performing a stability analysis, it was identified that the lower flanges of the scissors and the purlins of the roof did not meet the required standards. Based on the results and on the problems identified *in loco*, reinforcements and interventions were proposed for the roof: Installation of rigid ties, French hand and bracing; replacement of damaged profiles and correction of the painting system. These interventions were described in the project and have already been carried out.

Keywords: Coverage. Reinforcement. Steel. Arch. Corrosion.

1. INTRODUCTION

The effectiveness of a building executed in steel structure is obtained through the combination between steel quality, project preparation and execution. CARVALHO, et al (2017) highlights that the easy handling of the profiles that provides agility during the service, the good durability of the structure and the possibility of compatibility with other materials, such as concrete, are relevant factors that justify the demand for steel.

Due to the fact that steel structures are built by the association between metal plates and profiles through welded or bolted connections, it is necessary to correctly follow, during the execution, the assembly procedures determined in project, to ensure the performance and stability of the set (CARVALHO; FERNANDES; COSTA, 2017). The positioning of the parts and the properties of steel, such as fragility, resilience, ductility, hardness, toughness and fatigue are the conditions for the safety and durability of the structure.

Having known the reference parameters of the material behavior, as well as the dimensioning constraints of the corresponding standards, it is necessary to understand the agents responsible for the occurrence of pathological manifestations. Many structures present, with little time of use, corrosive manifestations mainly in the region of the connections and in the points of union between the metal profiles and the reinforced concrete structures, in the example of mixed structures. Environmental aggression, thermal conditions, techniques used during execution and lack of maintenance are the main causes for the occurrence of corrosion. In many cases, this does not happen only due to the inefficiency of the painting system, or the lack of maintenance, but it can also be driven by the arrangement of the elements in project. During the conception of the architecture and the modeling of the structure it is important to avoid regions where water can accumulate, depending on both the positioning of the profiles and how the connections will be executed, for example, the type of welding. The design must consider the environmental conditions and contribute to the structure's protection against corrosion. This involves the choice of profile types, their arrangement, and the type of connection. One must avoid the formation of gaps, concavities that allow water accumulation, contact with humid regions, parts that are difficult to see, or access for maintenance, etc. In addition, steel structures require an active maintenance system, both preventive and corrective, because when the presence of corrosion is identified in the structure the process cannot be allowed to spread until the system collapses.

Another pathology frequently identified in metallic structures is buckling, manifested by deformation of the profiles. This is the instability of the piece occurred by the incidence of compression stress, in which the element loses its straight shape and balance. According to Castro (1999), this type of collapse occurs, generally, due to the use of slender pieces and the little presence of brakes in the structure in the project conception. It can be observed that having a detailed knowledge about the requirements of the referring norms is crucial for the modeling and sizing process of the parts. The project conception phase must consider all the criteria required by the standards, especially regarding the limits of deformation and the relationship between resistant and internal forces, since these are the criteria that ensure the stability of the structure. It is up to the designer to evaluate which are the best resources to be used, considering the forces acting on the structure, the size of the building, its functionality and the environmental conditions of the place where it is executed. ROMAN (2016) calls this process of analyzing design variables during modeling structural optimization.

Therefore, this research is a case study of the roof of the sports gymnasium belonging to the Federal Institute of Education, Science and Technology of Tocantins (IFTO) - in the city of Pedro Afonso in Tocantins, which, due to the non completion of the project and the presence of pathologies in the structure, was not being used by the educational institution for safety reasons. The gymnasium in question belongs to a building that previously functioned as an agricultural school. The construction of the sports center was not completed, both the roof and the accommodations, such as changing rooms, bathrooms, etc.

The roof structure has 10 flat arched metal trusses supported on reinforced concrete pillars by bolted base plates. It has a total width of 32.30 m and length of 48.78 m. It presents the following elements: Shears or Plane Trusses, purlins, braces, rigid chains and ties. The structure was not being used because it had already been damaged due to pathologies and because the project had not been completed. This study performed a field reconnaissance of the structure executed until the time of the research, identifying possible disparities between the designed structure and the one executed on site, mapped the pathologies already present, and proposed elements of structural reinforcement in an intervention project. Figure 1 below is a record of the structure executed in the first moment.

Figure 1 - Sports Gymnasium Covering
Source: Author (2020)

The study to reinforce the roof of the Sports Gymnasium and the elaboration of an intervention project made it possible to reactivate the center and use it for the multi-sport activities proposed by the institution, also involving the community by offering a space for events.

2. METHODOLOGICAL PROCEDURES

2.1 Pathologies identified *in loco*

The identification of the pathologies present in the roof structure was performed by visiting the gymnasium and taking the necessary photographic records. The main(bpa)thology identified was corrosion, especially at the structural connection points. It was observ(ead) that all the lower flanges, diagonals and uprights that make up the first node of the scissors show a high level of corrosion, with paint peeling off, and some pieces are also broken. This factor indicates a possible accumulation of water in the region. Moreover, the plates used to make the connection between the steel structure and the concrete supports are, in general, degraded by corrosion, with reduction in the length of the pieces, especially those that form the first node of the bottom bar of the trusses. Figure 2 shows the records made during the on-site visit.

Figure 2 - Records of pathological manifestations in the pieces
Source: Author (2020)

It is possible to see that the geometry of the structure and the positioning of the profiles contribute to the accumulation of water at the joist support nodes on the concrete column. The entire inclination of the roof is directed to these regions, and the meeting of the flanges with the diagonals and uprights creates a space conducive to water storage. Furthermore, interaction points between different building systems, in this case the steel structure and reinforced concrete, are areas that require attention when painting, as the concrete can transfer moisture to the metal parts.

In general, the condition of the structure's profiles indicated that the pieces were not treated and painted properly, not all the necessary layers were applied, and that the painting system was not maintained. The elements that make up the highest parts of the roof do not present serious manifestations such as a reduction in the cross section or rust spreading; however, it was found that in some spots there is detachment of the painting system, probably due to poor preparation of the surface for the service. It was also observed that only one layer of protection was applied, probably of the type of finishing coat.

According to Castro (1999), uniform corrosion that causes the gradual loss of profile thickness is caused by the direct contact of carbon steel with the aggressive environment due to the absence of protection, inadequate protection or even lack of paint maintenance. It is noteworthy that project errors such as the inadequate arrangement of profiles, providing water accumulation, and the lack of drainage holes can be aggravating problems.

2.2 Project survey executed

The direction of the institute provided the project of the metallic structure of the gym, and it is dated November 2001. It was possible to observe that the project has little detailing of the structure, especially for the execution of the connections. There is no detail in the project that specifies how the connection of the elements in the structure should be made, the distance of the braces of the bracing, not even specification of welding points, their lengths and thicknesses. Another important detail that is not presented in the project is the positioning of the truss profiles in relation to the direction of the opening of the piece. It is understood that the lack of specification of the positioning of the lattice profiles may have contributed to the arrangement observed in loco.

During the visit to the gym, a survey of the executed structure was done to verify if there are divergences between the project initially proposed and the execution. A pachymeter was used to check the dimensions of the pieces and an ultrasound thickness gauge to check the thickness of the plates. The following divergences were identified:

a) The project indicates the installation of 44 purlins spaced at 94 cm (ninety-four centimeters). It is observed here a possible cause of instability in the designed structure, since to ensure such spacing the purlins dimensioned in the project are not supported on the nodes of the truss, offering less rigidity. On the other hand, the structure shows 27 purlins alternately supported (node yes, node no) in the nodes of the shear, being spaced in 1.47 m.

b) In the project no rigid ties are foreseen, however, during the visit it was verified the presence of rigid ties of angle bracket 50x50#2.3 mm braking the rigid chains attached to the central ridge of the roof. The ties were installed alternately between the truss spans.

c) The project does not indicate the bracing space, however, they are represented in a continuous way, without an open span. During the visit, it was observed that the bracing was installed with locking intervals every three thirds, with two open spans, i.e., not braced;

d) The project has 11 trusses, the first 3 of which are smaller and lower than the others. On the other hand, 10 trusses of the same size and height were executed.

e) In addition, the profiles used on site are not the same as those specified in the project; Table 1 below shows a comparison between the profiles determined in the project and those used on site.

Table 1 - Comparison between the profiles
Source: Author (2020)

Project	Execution
Lower and upper flanges - Simple foldedsteel U-profile 100x50#3.04 mm;	Lower and upper flanges - Simple folded steel U-profile 120x50#3.0 mm;
Diagonals and Mounting members - Simple folded steel U profile 92x40#2.28 mm;	Diagonals and Mounting members - Simple folded steel U profile 110x40#3.0 mm;
Tirds - U-profile bent steel stiffened 75x40x15#2.66 mm;	Tirds - U-profile folded steel stiffened 127x50x17#2.36 mm;
Rigid Chains - 3/8" diameter flat bar rolled steel profile	Rigid Chains - Folded steel angle profile 50x50#2.30 mm;
Tie rods - Not in the Project	Tie rods - Angled steel profile 50x50#2.30 mm;
Bracing - Rolled steel profile in flat rebar, 5/8" diameter	Bracing - Rolled steel profile in flat rebar with 10 mm diameter.

2.3 Structure modeling in Software

The modeling of the roof structure was done in CypeCad software. The roof was modeled on site according to the information collected during the visit, informing the proper steel characteristics, loads, combinations for the ultimate limit state and support conditions. The objective of the modeling was to perform a stability analysis of the roof. It should be noted here that the stability analysis performed considered the structure executed and inspected on site, since the design provisions were different from reality. Figure 3 below shows the modeling performed in CypeCad software.

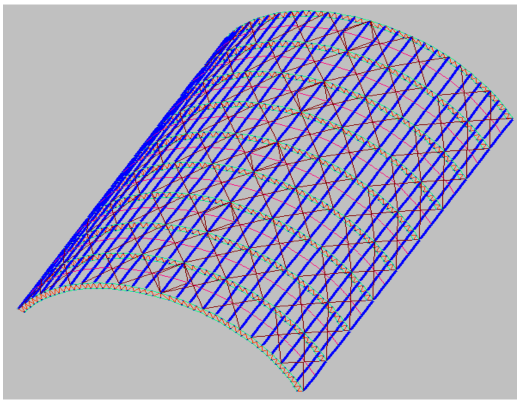

Figure 3 - Modeling of the executed roof seen from the right side
Source: Author (2020)

3. ANALYSIS OF RESULTS AND PROPOSALS FOR STRENGTHENING

After launching the structure, the stability of the roof was analyzed for the ultimate limit state (ULS), also in the Cype3D software, to identify the current safety situation of the gym. The bars identified in red in Figure 4 below were not successful in the verification according to the limitations established by NBR 14762 - Design of steel structures made of cold formed profiles, and ABNT NBR 8800 - Design and execution of steel structures in buildings (Limit State Methods). Weakness points were identified in the lower flanges of the trusses and in the purlins. According to the analysis of the results, the necessary interventions were proposed to solve the unmet verifications, resulting in a project for structural strengthening of the gymnasium roof. For the results to be presented in the next section, consider:

b – Element width;

t – Element thickness;

λ – Slenderness limit;

$N_{t,Sd}$ – Axial tensile force;

$N_{c,Sd}$ – Axial compressive force;

$N_{t,Rd}$ – Resistant axial force in tensile calculation;

$N_{c,Rd}$ – Axial compressive design resisting force;

V_{Sd} – The shear force of calculation;

V_{Rd} – The shear strength of calculation;

M_{Sd} – Requesting bending moment of calculation;

M_{Rd} – The design resistant bending moment.

T_{Sd} – The required torsional moment of calculation;

T_{Rd} – Torsional resistant moment of calculation.

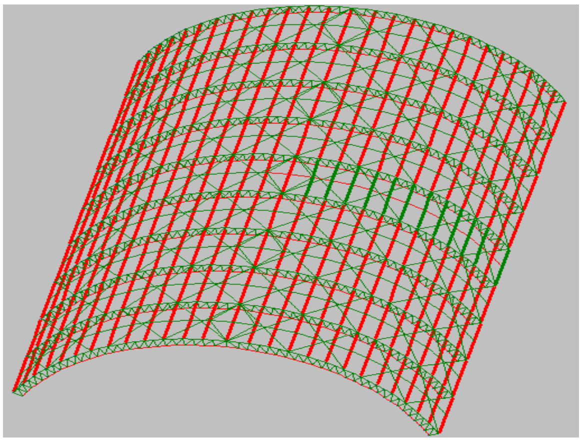

Figure 4 - Ultimate Limit State Verification
Source: Author (2020)

3.1 Lower flanges

The pieces did not meet the checks for Slenderness Limit and Compressive Strength. Table 2 below presents the results obtained in the analysis of one of the bottom flanges. The cells highlighted in red represent the checks that were not met.

Table 2 - Calculation results for the lower flange
Source: Author (2020)

<table>
<tr><td colspan="11">Lower flange (N86-N87) - Verifications according to ABNT NBR 14762 (2010)</td></tr>
<tr><td colspan="2">Length-Thickness Ratio</td><td colspan="2">Slenderness limit</td><td colspan="2">Traction</td><td colspan="2">Compression</td><td colspan="3">Cutter</td></tr>
<tr><td colspan="2">(b/t) ≤ 90</td><td colspan="2">λ < 200</td><td colspan="2">n (Nt,sd/Nt,rd) ≤ 1</td><td colspan="2">n (Nc,sd/Nc,rd) ≤ 1</td><td colspan="3">n (Vsd/Vrd) ≤ 1</td></tr>
<tr><td>b (mm)</td><td>108</td><td rowspan="2">λx</td><td rowspan="2">799</td><td>Nt,sd (t)</td><td>1.814</td><td>Nc,sd (t)</td><td>3.943</td><td colspan="2">Vx</td><td>Vy</td></tr>
<tr><td>t (mm)</td><td>3</td><td>Nt,rd (t)</td><td>17.518</td><td>Nc,rd (t)</td><td>0.145</td><td>Vsd (t)</td><td>0.002</td><td rowspan="3">No requesting effort</td></tr>
<tr><td rowspan="2">(b/t)</td><td rowspan="2">36</td><td rowspan="2">λy</td><td rowspan="2">45.9</td><td rowspan="2">n</td><td rowspan="2">0.104</td><td rowspan="2">n</td><td rowspan="2">27.2</td><td>Vrd (t)</td><td>2.202</td></tr>
<tr><td>n</td><td>0.001</td></tr>
</table>

The slenderness index is a ratio obtained as a function of the radius of gyration of the profile section in the direction of the analyzed axis, the type of support and connection at the bar node, and the distance between the braces of the part. This distance between the interlocks is called the buckling length, which is the length that a given bar has to buckle according to the movement limitations of its links at the nodes. This restraint prevents the part from excessive deformation and loss of strength and stability. Thus, the length of buckling is directly proportional to the slenderness index, the higher it is, the slimmer the part becomes and it is free to deform.

In the case of the gymnasium roof analyzed in this study, the scissor structure does not present any interlocking for the lower chords in order to reduce the free length for the bending of the elements. Thus, the buckling length for the lower flanges of the scissor beams is limited by the spacing between the concrete supports at the ends of the truss, totaling a value of 37.06 m, not enough to limit the bending of the part around the axis, as required by the standard, as evidenced in Figure 5 below.

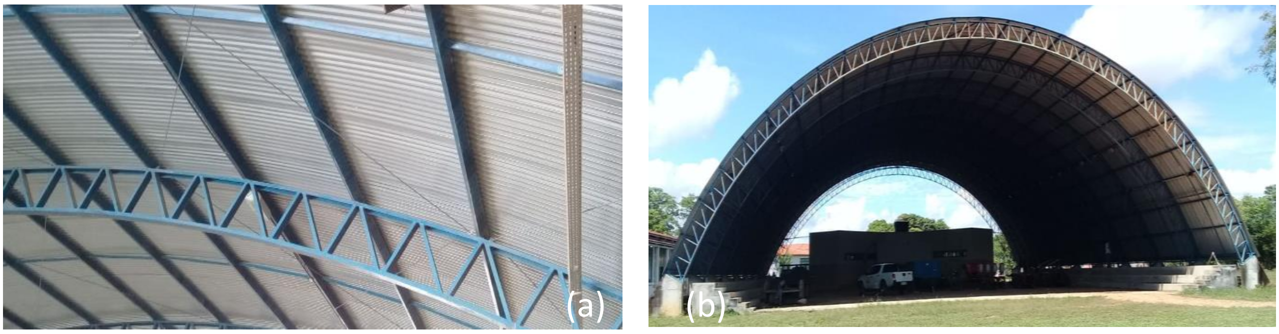

Figure 5 - Registration of the truss bottom flanges
Source: Author (2020)

Similarly, the greater the value of the effective length of buckling in the axis, the lower the compressive strength of the part. Given the above, it is observed that the absence of interlocking in the bottom flanges of the trusses is the factor responsible for the ineffectiveness in verifying the slenderness index and the compressive strength of the pieces.

For both identified problems, it was proposed the instal(lbat)ion of angle brackets made of bent steel profile 50x50#2.30 mm in A36 steel, in the lower flanges below each truss. One flange of the French hand was locked below the bottom flange, and the other flange on the steel joist at a distance of 80 cm from the connection node between the joist and the upper flange (oaf) the truss. In this way the buckling length of the bottom chords became the distance between the purlins, which are spaced every 1.47 m, a much smaller space than the original. This reduction provides a reduction in the slenderness index of the part and increases its compressive strength.

After inserting the angle brackets, the checks were performed according to the limitations set forth in the standard. Figure 6 below shows the modeling of the installation of the angle brackets on the structure.

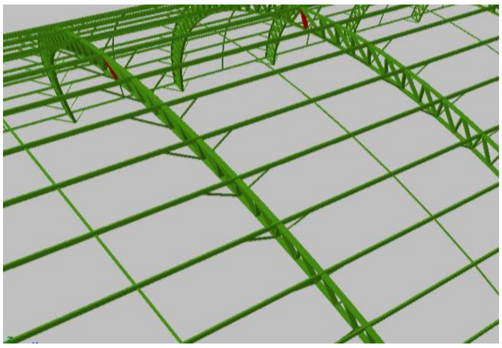

Figure 6 - Representation of the installation of French hands
Source: Author (2020)

3.2 Purlins

In general, the purlin profiles did not resist bending and flexural tensile loads, and some pieces were also unsuccessful in the flexural-compression test. Table 3 below presents the calculation results obtained for one of the pieces. The cells highlighted in red represent the verification not met.

Table 3 - Calculation results for metal purlins
Source: Author (2020)

Purlin (N943-N1297) - Verifications according to ABNT NBR 14762 (2010)			
Length-Thickness Ratio		Slenderness limit	Traction
(b/t) soul≤ 500	(b/t) table ≤ 60	$\lambda < 200$	n (Nt,sd/Nt,rd) ≤1

b (mm)	117.56	b (mm)	40.56	λx	109.5	Nt,sd (t)		0.035
t (mm)	2.36	t (mm)	2.36			Nt,rd (t)		16.097
(b/t)	49.81	(b/t)	17.19	λy	147.4	n		0.002
Cutter				Bending + Shear				
n (Vsd/Vrd) ≤ 1				$(Msd/M0,rd)^2+(Vsd,V0,rd)^2 \le 1$				
Vx		Vy		MxVy				MyVx
Vsd (t)	0.003	Vsd (t)	0.334	Msd (t*m)	0.451	Vsd (t)	0.025	No requesting effort
Vrd (t)	1.597	Vrd (t)	4.628	M0,rd (t*m)	0.621	V0,rd (t)	4.628	
n	0.002	n	0.072	n		0.527		
Flexo-compression						Flexion		
(Nc,sd/Nc,rd)+ (Mx,sd/Mx,rd)+(My,sd/My,rd)≤1						n(Msd/Mrd) ≤1		
Nc,sd (t)	0.003	Mx,sd (t*m)	0.449	My,sd (t*m)	0.002	Mx		My
Nc,rd (t)	3.475	Mx,rd (t*m)	0.335	My,rd (t*m)	0.16	Msd (t*m)	0.454	No requesting effort
n			1.354			Mrd (t*m)	0.335	
						n	1.36	
Flexo-traction						Compression		
(Nt,sd/Nt,rd)+ (Mx,sd/Mx,rd)+(My,sd/My,rd)≤1						n (Nc,sd/Nc,rd) ≤ 1		
Nt,sd (t)	0.013	Mx,sd (t*m)	0.454	My,sd (t*m)	0.002	Nc,sd (t)	0.004	
Nt,rd (t)	16.1	Mx,rd (t*m)	0.335	My,rd (t*m)	0.156	Nc,rd (t)	3.475	
n			1.369			n	0.001	

The installation of the French hands proposed to solve the instability of the bottom flanges, also helped in solving the problem of the purlins, since the locking of the French hand is done with a tab attached to the bottom flange of the truss and the other tab attached to the purlins. In this way the unbraced length of the purlins is reduced, thus decreasing the bending moments generated in the bar in the direction of its length axis. Figure 7 below shows a comparison between the lengths of the purlins in a span between two shears, before and after installing the French braces.

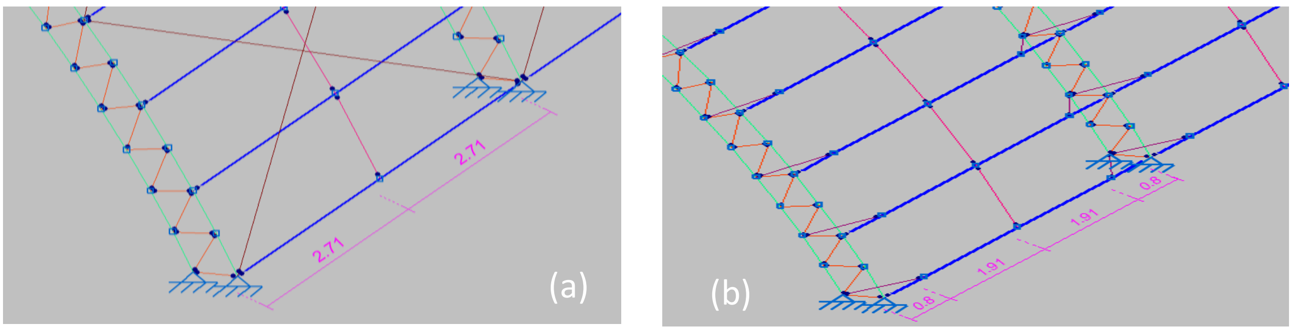

Figure 7 - Comparison of the lengths of the purlins before and after the intervention Source: Author (2020)

By reducing the requesting moments in the purlins it was possible to solve the three non-conformities presented by the elements, according to the standards, since both unmet verifications are calculated as a function of the requesting bending moment. Thus being reduced the loads, the bending resistance of the pieces became sufficient to support the request.

At the construction site, initially, rigid ties of folded steel angle profile 50x50#2.30 mm were installed, locking the connection between the rigid chains and the purlins at the highest point of the shears. These were installed alternately between the spans of the trusses. It was proposed the installation of these ties in the spans where they were not previously executed, considering that this element helps in increasing the rigidity of the locking of the rigid chains with the purlins at the highest point of the arch, and consequently more requested. It is a contribution in the intervention project to increase the rigidity of the structure and prevent the lateral displacement of these purlins. Moreover, it was observed during the on site visit that due to the non completion of the work, some bracing was not installed, leaving the structure, during this time, with unlocked elements. The project indicated that these missing bracing elements should be installed, as well as the installation of new elements in the spans for which they were not initially proposed. The installation of these bracing elements reduces the buckling length of the upper flanges of the shears where they are attached, offering greater stiffness to the structure in an area that was prone to instability for a long time.

3.3 Other interventions

According to the analyses made, an intervention project was prepared detailing the elements to be added to the structure and the necessary interventions according to the degree of impairment of the profiles. For all the profiles used in the elements to be inserted, ASTM - A36 steel was used, due to market availability and compatibility with the profiles of the structure already in place.

Due to the state of degradation of the parts that make up the first node of the trusses, it was determined in the project that all the lower flanges, diagonals and uprights that make up the first (lower) node of the scissor be replaced. The base plates and lateral connection, and the connection bolts between the steel structure and the concrete support were also changed. As all the trusses had these parts damaged, the project determined that the elements would be replaced one truss at a time, and one side at a time, to avoid extreme movements in the structure during the execution of the service.

Furthermore, since it was identified that this area is prone to water accumulation, due to the geometry of the roof and the arrangement of the profiles, concreting was performed until the formation of the second shear knot, as an extension of the support column. The concrete to be used has no structural function, it served only as a protection layer for the vulnerable area. To improve the adherence between the concrete and the steel pieces, it was indicated to install welded round bars with 8 mm diameter and 16 cm length, in the metallic profiles, obeying a 10 cm spacing between the bars. These rebars act as staples inside the concrete improving the interaction with the metallic structure. Drainage holes were also drilled in the lattice pieces immediately above the concrete area, to prevent the accumulation of water in the profiles. These determinations were exposed in the project with explanatory drawings.

As for the treatment of the parts not compromised by corrosion, it was proposed to correct the paint system due to the identification of some points of detachment of the paint film, also considering the time that the cover has not received maintenance since it was started. According to the recommendations of the 2017 Design and Durability Manual of the Brazilian Center for Steel Construction (CBCA), since the paint system on the other parts of the roof, those that have not been replaced, is superficial, manual cleaning to remove rust was performed using a steel brush and sanding.

The painting system was chosen according to ISO 12944 (2017) - Protection by means of anticorrosive painting systems. In accordance with the specifications of the standard, a St.2 (ISO 12944) cleaning system (ISO 12944) was determined in the design, where Alkyd resin primer should be applied in two coats with a dry thickness of 80 µm. The intermediate layer of the system and the finishing layer must also be made with

Alkyd Resin material, with the intermediate layer being made in two coats with a dry thickness of 120 µm, and the finishing layer in two coats of 200 µm. There are 1606 m² of area for the execution of the paint protection system.

3.4 Checking the Connections

Given the interventions proposed in the project, there were connections to be checked and detailed. A spreadsheet was prepared to map all the welds that would be executed in the construction site. It was mapped as follows:

Detail 01 - Replacement of the compromised lattice profiles, replacement of the connecting elements with the concrete pillars and concreting up to the second lattice node; **Detail 02** - French hand installation; Detail **03** - bracing installation; **Detail 04** - installation of the rigid ties.

Thus, an identification of the meeting points of the plates in each mapped detail was performed to determine the thickness of the welding leg. They were dimensioned according to NBR 8800 (2008). In all connections executed, the length of the weld bead is equal to the entire meeting edge between the two profiles, this is also an alternative to avoid cracks that promote the accumulation of water, opting therefore for a continuous weld. After all the verifications were made according to the efforts calculated in software, the interventions were arranged in the project specifying the welding points, welding type, thickness and electrode, with quantitative material and explanatory notes. The detailing of the welds in the project followed the provisions of ABNT NBR 7165 (1982) - Graphical welding symbols for ship and railway construction. The dean's office of the IFTO - Tocantins Federal Institute of Science and Technology Education was responsible for preparing the budget spreadsheet, as well as for the other projects required to complete the work, such as renovating the changing rooms, installing the bathrooms, among others. A bidding process was elaborated, and the construction company Sambaiba Construções Ltda was responsible for the execution of the remodeling project. The Figure 8 below shws the reinforcements installed in the roof and the other interventions.

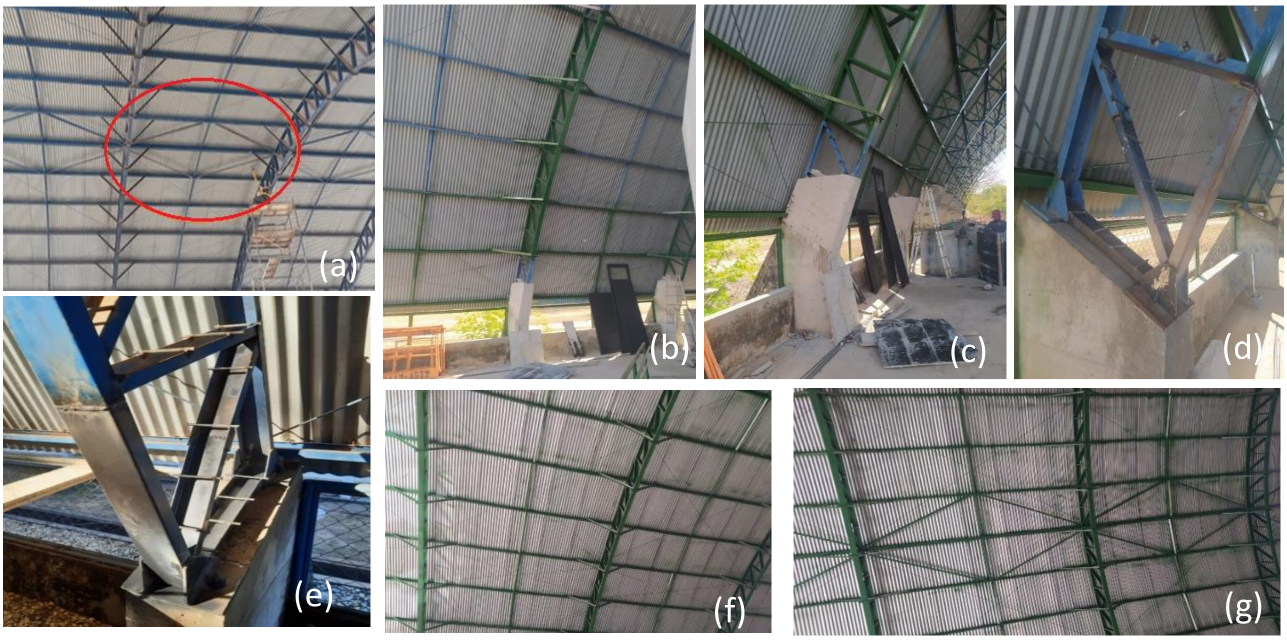

Figure 1 - Interventions performed on the roof Source:
Author (2020)

4. FINAL CONSIDERATIONS

The study developed allowed us to understand that the pathologies that develop in a steel structure can originate from a combination of factors that involve from the project conception phase to the execution and post-delivery maintenance of the work. It was observed that it is important that the design engineer has knowledge about the possibilities in which a corrosion pile can occur, in order to avoid that in the details of his project these situations are created. Likewise, during the conception of the roof elements it is necessary to consider the calculation variables required by the sizing norms, so that the structure is optimized to meet these criteria.

The gymnasium analyzed in this case study is an example of all these factors. The initial design had few explanatory details for the construction team, which probably caused the disparities observed in loco between design and execution. Moreover, it was observed that there was not much concern in avoiding regions of water accumulation, and the arrangement chosen for the profiles contributed greatly to the structure's compromised state.

In view of all the analyses carried out, it was possible to propose an intervention project that would provide stability to the roof of the space, which was previously unused. The project included the insertion of new pieces, the replacement of compromised profiles and connections, and a new painting system. The roofing project intervention was one of the contributors to the general renovation project of the gymnasium, prepared by the Federal Institute of Education, Science and Technology of Tocantins. It is suggested that further research be conducted to present in detail the role of each roof restraint element, tie-rods, bracing, braces and rigid chains in the verification required by design codes. These studies will contribute to the designers in the future phases of project conception.

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