TECHNICAL STUDY OF REINFORCED CONCRETE BUILDINGS OF DIFFERENT HEIGHTS: ANALYSIS OF THE QUANTITY OF CONCRETE, STEEL AND FORMWORK

ALVES, Heytor Donizetti Pereira
Universidade do Estado de Mato Grosso (UNEMAT).
heytoralves7@gmail.com

PINHEIRO, Roberto Vasconcelos
Universidade do Estado de Mato Grosso (UNEMAT).
rpinheiro@unemat-net.br

MASCARENHAS, Fernando Júnior Resende
Universidade Federal de São Carlos (UFSCar).
fer.jr.resende@hotmail.com

CHRISTOFORO, André Luis
Universidade Federal de São Carlos (UFSCar).
christoforoal@yahoo.com.br

ROCCO LAHR, Francisco Antônio
Universidade de São Paulo (USP).
frocco@sc.usp.br

ABSTRACT

The purpose of this study was to analyze the technical characteristics of buildings from three to 21 pavements, with a ribbed slab, as well as using different values of characteristic resistance to concrete compression (f_{ck}), varying from 25 MPa to 40 MPa, with a geometric relation in plan, of 1:1. Utilizing the obtained results, it is possible to contribute to the structural design, in the dimensioning, and to contribute to the budgeting of the structure of buildings in reinforced concrete with the same geometric relation. Data collection and organization were obtained through reinforced concrete structure analysis software and spreadsheets. The procedures applied were divided into architectural design, structural design, global stability analysis, quantification of inputs, and definition of technical parameters. The analysis of the results concluded that: the increase of f_{ck} caused a great reduction in steel consumption. With the increase in the number of pavements type, there was an expressive increase in the consumption of the beams, the buildings of smaller height did not present a great reduction in the consumption of concrete with the increase of f_{ck}, if compared with the buildings of greater heights.

Keywords: Reinforced Concrete Structures. Ribbed Slab. Technical Analysis.

1. INTRODUCTION

Due to the growing urbanization processes that large cities are currently experiencing, consequently, there has been high population growth, and with the increase in this migrant population, there has also been a need for new housing. However, the traditional construction of the horizontals homes is no longer able to support a large number of people, so it requires an extensive area to attend to this high demand. This makes it inefficient in this respect, as it does not meet the necessary financial return, since the land price has undergone great increases due to real estate speculation. Hence, vertical construction techniques have been used.

The first reinforced concrete building in Brazil was the building Joseph Gire, which is located in Rio de Janeiro, had its construction started in 1927, and it was opened in 1930. It has 22 floors and 102 meters high, according to Heloíza Gomes, in Veja Rio magazine's interview (Veja, 2018). This movement of vertical constructions has been intensifying more and more, and in the current conjuncture, Brazil is in the trend of verticalization. According to surveys by the Brazilian Institute of Geography and Statistics (IBGE, 2017), the 2010 population census showed that the number of residents living in apartments has almost doubled in a decade, and currently, 1 in 10 Brazilians live in apartments. In the northern region of the country, this number is even higher, where the growth rate of apartments grew 3.5 times higher when compared to the rest of Brazil.

In this context, reinforced concrete buildings are inserted, in which increasingly tall buildings are sought, as this makes them more attractive. This was possible due to the advancement of engineering technology with new software for modeling and structural calculations, together with new technologies for structures, for example, as concretes with greater compressive characteristic resistance to compression (f_{ck}).

In reinforced concrete buildings, the increase of the characteristic compressive strength of concrete allows the reduction of the cross-section of structural elements, hence, decreasing the concrete consumption. Although there are advantages, it must be verified if the reduction of concrete consumption is economically viable, because concretes with higher strength have a higher price.

Hence, this work aims to investigate a hypothetical building with ribbed slabs and geometrical relation in plant of 1:1, and characteristic compressive strength of concrete, f_{ck}, of 25, 30, 35, and 40 MPa. Based on that, the results of the number of inputs (concrete, steel, and formwork) of the superstructure are presented and analyzed. The building of this study is made of the typical floor with the following variations of repetitions: 3, 5, 7, 10, 13, 15, 18, and 21 pavements.

2. THEORETICAL FOUNDATION

2.1. STANDARDIZATION

For the elaboration of this project in the structural conceptional, analysis, and design aspects, certain standardizing documents from the Brazilian Association of Technical Standards (ABNT) are used. The used codes are ABNT NBR 6118:2014 – Project of reinforced concrete structures – Procedure, ABNT NBR 6120:1980 – Loads for calculation of buildings structures, ABNT NBR 6123:2013 – Forces due to wind in buildings, ABNT NBR 7480:1996 – Bars and steel wires for reinforcement for reinforced concrete, ABNT NBR 8681:2003 – Actions and safety in structures– Procedure, ABNT NBR 8953:2015 – Concrete for structural purposes – Classification by specific mass, by resistance and consistency groups, ABNT NBR 14931:2004 – Execution of concrete structures – Procedure.

2.2. REINFORCED CONCRETE

The reinforced concrete is the result of the association of simple concrete with steel bars. This association is due to the low tensile strength of concrete, which is about 4 to 8 % of the compressive strength. Therefore, the steel bars have the function of resisting the effects of tensile and increase the load capacity of the compressed parts (Araújo, 2014).

According to Fusco (2008), the simple concrete commonly used has considerable resistance to compression f_{ck}, and it is around 20 to 50 MPa. According to ABNT NBR 6118:2014, it is possible to have simple concretes with f_{ck} up to 90 MPa.

Due to adherence between concrete and steel, their deformations are practically the same. The maximum allowable strain of the reinforcement is 10‰ (10 per thousand) to avoid the existence of plastic deformations. Also, the maximum deformation of the concrete under bending is 3,5‰ (3,5 per thousand) to avoid crushing by compression of the concrete. Another function of concrete is also to protect the reinforcement from corrosion and fire (Araújo, 2014).

Concrete is a material that presents elastics and inelastic deformations when it is loaded, and deformations by fluency, shrinkage by drying, or by thermal contraction. Fluency is defined as the continuous increase in deformations for a given stress. Drying shrinkage is the process of decreasing the volume of concrete associated with moisture loss during curing. The thermal deformation, which is associated with the coefficient of thermal variation of the material, occurs due to the temperature variation, and it is relevant when there is a high volume of concrete (Pinheiro, 2007).

Concrete has been used all over the world due to its benefits, for example, its economy, conservation, adaptability, faster construction, fire safety, impermeability, and shocks and vibrations resistance (Bastos, 2006).

2.3. STRUCTURAL CONCEPTION

According to Pinheiro (2007), the structural conception consists of choosing the structural system that integrates the resistant part of the building structure. This is one of the most important steps because it is in this stage that the choice and location of the structural elements will be made so that it creates a structure capable of resisting efforts. There are several structural models, each one with its advantages and disadvantages.

Araújo (2014) advocates that the “conventional” structural system is formed by slabs, beams, and columns (pillars) or by the union of these elements. In this system, the load path occurs through the slabs, which receive the loads from the building and transmit them to the beams. The beams have the function of receiving the vertical loads from the slabs and walls and transmitting them to the columns and, they transmit the loads to the foundation. There are also systems where there are no beams, so the slabs are supported directly on the columns.

Carvalho and Pinheiro (2009) state that a slab without beams system has numerous advantages compared to the conventional ones: simplification of concreting, simplification of reinforcement, reduction of execution time and costs, simplification of building installations, improvement of final quality, and reduction of coverings, and reduction of the total height of buildings. However, there is the possibility of happening punching of the slab-column due to the transverse displacement of the slabs, instability of the building to the lateral actions (loads), and huger consumption of concrete and steel.

One structural system with slabs without beams is the flat slabs. In this structural system, the slabs are directly supported by the columns, without the presence of beams or caps. Its thickness must be sufficient to guarantee its resistance to punching, shear, and bending. It is recommended to use support beams on the edges to avoid the application of linear loads of the walls on the free edges of the slab. Besides, the edge beams can avoid problems of the puncture on the edge and corner pillars. The use of this model has certain advantages such as it facilitates the passage of the ducts under the underside; the formworks are simpler and more economical; it is easier to set up and concreting (Araújo, 2014).

Regarding this structural system, it is also important to mention the ribbed and waffle slabs. They are composed of a set of beams joined by the table, and they have a structural behavior between the solid slab and the grid. The adoption of this slab model allows industrialization of the process, increasing productivity and reducing losses (Pinheiro, 2014).

Ribbed slabs are 50% thicker when compared to solid slabs; however, they are used to reduce the structure's own weight (dead load). This is because the spaces between the grids can be filled with inert materials and those with a low specific weight. Moreover, the resistance and tensile strength of this type of slabs are concentrated in the region of the grids (Araújo, 2014).

2.4. ACTIONS IN THE STRUCTURE

According to ABNT NBR 6118:2014, to perform the structural analysis it must first take into consideration all the actions that will affect the structural security. ABNT NBR 8681:2003 defines actions or loads as "causes that provoke efforts or deformations in the structure. From a practical point of view, the forces and deformations imposed by the actions are considered as if they were the actions themselves". It also classifies these actions as permanent, variable, and exceptional loads (ABNT, 2014). The permanent actions are those that occur during the whole useful life of the structure with little variability or even being able to grow over time tending to a limit. They can be divided into direct, which can be the element's own weight and, indirect, which can be the retraction and fluency of the concrete (Araújo, 2014).

The variable actions are those that happen with expressive variability during the useful life. They are classified as normal and special according to their probability of occurrence. The first one has a high possibility of occurrence, and its consideration in a project is mandatory, while the second one is defined for specific occasions (Araújo, 2014).

Exceptional actions are those that have a very low probability of occurrence and a very short duration over the useful life, however, in some structures, they must be considered (Araújo, 2014).

The loads acting on the slabs are predominantly applied perpendicular to their own plans. The loads applied to this element are mainly the permanent weight of the structure itself, coverings, and walls of the building (dead load). The live actions on slabs must be established according to ABNT NBR 6120:1980, and they depend on the type of use of the building.

The columns receive the entire load from the pavements and, according to ABNT 6120:1980, in commercial and residential buildings not used as building supplies, live loads can be reduced based on the values present in table 4 of ABNT 6120:1980. The columns are also subjected to horizontal efforts caused by the action of the wind on the structure, and the guidelines for calculating this action can be seen in ABNT NBR 6123:2013. According to the geographic location of the project, it is established the basic wind speed, V0, using the isopleth map.

2.5. SECURITY AND LIMIT STATES

ABNT NBR 6118:2014 states that concrete structures must meet three criteria: strength, service performance, and durability. First, the resistant capacity is responsible for the safety of the rupture of the structure. Second, the performance in service is the ability of the structure to remain in conditions of use without presenting damages that compromise its use during its useful life. Third, durability is the capacity to resist environmental actions, which are defined during the preparation of the project.

ABNT NBR 8681:2003 also presents the security parameters of reinforced concrete buildings, and it determines the load factors for the combinations of the loads and the determination of the design loads that will be used to verify the limit states.

To investigate these parameters, there are some calculation techniques, which are the limit states, and they are separated into ultimate and service limit states. The ultimate limit state (ULS) refers to the collapse or ruin that causes total or partial paralysis of the structure. The service limit states (SLS) is the one that makes the use of the structure impaired, and in usual reinforced concrete structures the limit states of excessive deformation and crack opening are verified (Araújo, 2014).

2.6. STRUCTURAL ANALYSIS AND OVERALL STABILITY

According to ABNT NBR 6118:2014, the structural analysis aims at the determination of action impacts on the structure to ascertain the ultimate and service limit states. It also allows the distribution of efforts, tensions, and deformations.

For the analysis of the overall stability of the structure, it is necessary to know the type of structure, whether it has fixed nodes or mobile ones. ABNT NBR 6118: 2014 defines a structure as of fixed node “when the horizontal displacements of the nodes are small and, as a result, the global 2nd order effects are negligible (less than 10% of the respective 1st order efforts)”. ABNT NBR 6118: 2014 also defines structures as mobile node ones “when the horizontal displacements of the nodes are not small and as a result, 2nd order effects are important (greater than 10% of the respective 1st order efforts)”.

To know exactly what are 2^{nd} order efforts, it is necessary to know the phenomenon that originates them. Geometric nonlinearity is the effect originated from the simultaneous actions of horizontal and vertical loads that cause lateral displacement in the nodes. This generates a balance of the structure after its deformation, creating new soliciting efforts, called 2nd order effects (Giongo, 2007).

For the analysis of the overall stability of the structure, checks can be made to find out whether it is necessary or unnecessary to consider global 2nd order efforts. For that, the instability parameter "α" (qualitative effect) and the coefficient "γ_z" (quantitative coefficient) must be analyzed, according to ABNT NBR 6118: 2014, items 15.5.2 and 15.5.3.

The calculation of the instability parameter "α" takes into consideration several factors, such as the total height of the building; the stiffness of the columns, which has a limit value being $\alpha = 0.6$ prescribed for $n \geq 4$ for the usual building structures (where n is the number of floors of the structure). If the limit value is met, the structure will present global 2nd order efforts not greater than 10% of the first effects on the structure (Araújo, 2014).

According to Moncayo (2011), the coefficient "γ_z" qualifies in a simplified and very efficient way the overall stability of reinforced concrete buildings and determines the second-order efforts by simply increasing first order efforts.

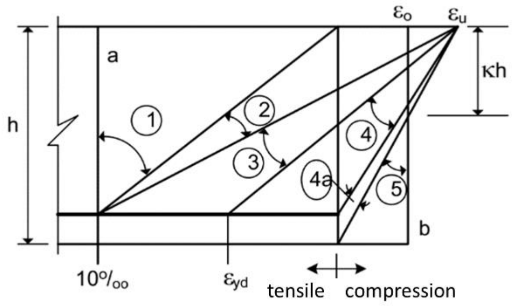
Carvalho e Pinheiro (2009) also point out that the evaluation of the "γ_z" coefficient of the final global efforts is valid for reticulated structures of at least four floors cannot be hugger than 1.3. The structure is considered to be of fixed nodes when $\gamma_z \leq 1.1$.

2.7. DESIGN OF STRUCTURAL ELEMENTS

ABNT NBR 6118:2014 explains that after the structural analysis, three steps must be investigated: design, verification, and detailing. These steps aim to ensure safety concerning the limit states of the entire structure or in each of its parts. For security to be met, the requesting efforts must not exceed the resistant efforts for all limit states.

According to Bastos (2006), figure 1 represents how the ultimate limit state can occur. It can be caused by excessive plastic deformation of the reinforcement, which is represented by domains 1, 2, and 3; but it can also be caused by excessive shortening of the concrete represented by domains 3, 4, and 5.

Figure 1: Dimensioning domain



Source: Araújo (2014).

Araújo (2014) remarks that figure 1 can be used to classify reinforced concrete elements in Under-Reinforced, Balanced and Over-Reinforced. A structural element is called under-reinforced when the reinforcement rates are lower than necessary (domain 2), and the failure happens by the excessive deformation of the reinforcement without crushing the concrete. On the other hand, balanced structures are those when the reinforcement rate is consistent with the applied loads, and the failure occurs in domain 3 due to the crushing of concrete and yield of the reinforcement. In this stage, the structure presents signs of warning for intense cracks before the rupture. Finally, the over-reinforced structures have a very high reinforcement rate concerning what is necessary, and their rupture happens in domain 4, where the reinforcement does not yield and breaks due to the crushing of the concrete, there are no warnings, and the failure is due to fragile rupture. Because of this deformation diagram, the design will be well done if it is in domain 3, where it is possible to take advantage of all the strength of concrete and steel.

2.8. DESIGN OF STRUCTURAL ELEMENTS

Regarding the technical-economic analysis of reinforced concrete structures, Costa (2012) made an analysis of two structural systems for a residential building in reinforced concrete, aiming to estimate the consumption of materials. One structural system had 4.0 meters (m) of the distance between the columns and the frames, and the other one had 6.5 m of distance, using the same architecture. The author obtained as result savings of 12% in the cost for the structure with a smaller span (Costa, 2012).

Spohr (2008) analyzed an edification with 10 pavements and two types of structural systems. One system was the conventional one made of columns, beams, and massive slabs, and another one was made of ribbed slabs that were supported in a pillar. The author noticed that when using the ribbed slab system, there was an 18.1% cost reduction compared to the conventional solid slab system.

Silva e Souza and Lopes (2016) also examined the conventional structural system formed by massive slabs and a system composed of waffle slabs. The number of floors of the analyzed building was not informed by the authors. The result found was that there was a reduction of 24.4% in the cost of the work when using the waffle slab system compared to the conventional one.

Lanini et al. (2019) analyzed structures from 3 to 21 pavements, with geometric projection in plain of 1:4, varying the value of the compressive strength of concrete, with increasing compressive strength. The authors made a comparison of input consumption and concluded that with the increase in compressive strength, the reduction in steel consumption in slabs and beams was 2.5% and 6.5%, respectively, while in columns there was an average reduction of 19% (Lanini et al., 2016).

Pillon (2017) carried out another analysis of a concrete structure, ranging from 3 to 21 pavements, with an increase in the compressive strength of concrete from 25 MPa to 40 MPa, with a geometric projection of 1:1 and dimensions of 20 x 20 m. Based on that, they conclude that with the increase in the compressive strength of concrete, there was a significant reduction in the steel consumption of the columns. Certain columns had a reduction of around 37.32% in steel consumption.

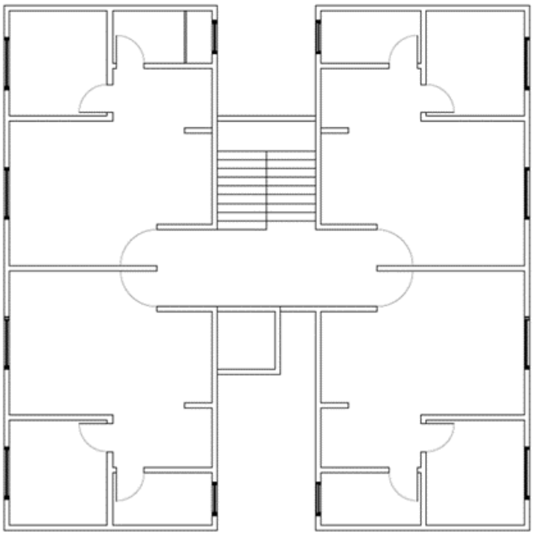
3. METHODOLOGY

This research was developed in five steps: 1) definition of the architectural project, respecting the geometric ratio of 1:1 in plane, with the dimensions of 15 m x 15 m; 2) structural conception and definition of the loads acting on the structure; 3) design and verification of structural elements in terms of limit and overall stability; 4) quantification of inputs (concrete, steel, and formwork; 5) definition of the technical parameters of the carried out project.

3.1. FIRST STEP – ARCHITECTURAL PROJECT

The architectonic project that was used presents the geometric ratio in the 1:1 plane, with dimensions of 15 x 15 m, according to figure 2, and with a high of 3 m between the pavements.

Figure 2: Architectonic project



Source: Authors (2018).

Eight types of buildings, which are named A, B, C, D, E, F, G, and H, are analyzed with different amounts of repetitions of the typical floor varying from 3 to 21 pavements, according to table 1.

Table 1: Buildings proposed concerning the number of pavements

Number of pavements	Height (m)	Nomenclature
3	9	A
5	15	B
7	21	C
10	30	D
13	39	E
15	45	F
18	54	G
21	63	H

3.2. SECOND STEP –STRUCTURAL CONCEPTION AND LOADS ACTING ON THE STRUCTURES

The structural conception was idealized respecting the restrictions imposed by the architectural project, as well as the premises to guarantee the overall stability of the building.

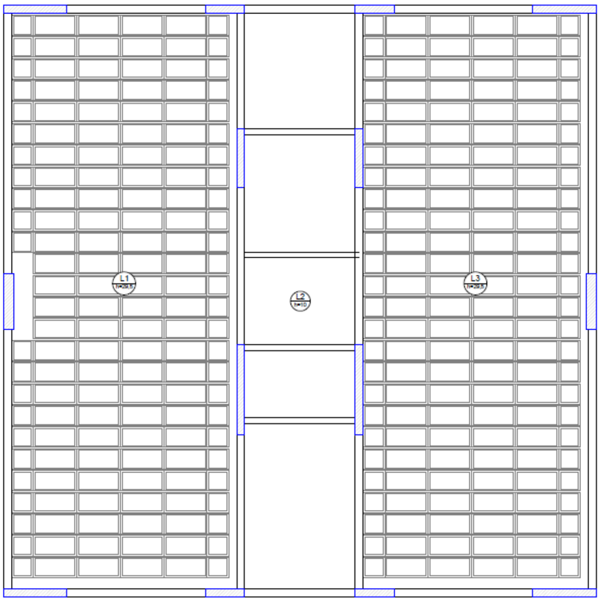
The columns were initially connected in three and four lines, in the “x” and “y”, respectively. Subsequently, the beams were located on the outside of the building and thus delimited the outline of the slabs.

Due to its dimensions, one-way joist slabs (ribbed slabs) were adopted because it was found that this model was better when compared to the waffle slab. The plastic bucket form adopted had the dimensions of 100 cm x 55 cm with a 10 cm wide and 22.5 cm high rib and a

7 cm thick slab cover. Thus, totaling a slab with a thickness of 29.5 cm. The shape plans of the slabs of the proposed buildings, in general, have the geometric characteristics shown in figure 3.

Due to the architectural restrictions, the overall slab is made of 2 planes of ribbed slabs, which contemplated the interior of the apartments, and a massive slab (because it is a small span), which contemplated the circulation area of the building. Figure 3 shows the representation of the structure design.

Figure 3: Structural Conception of Buildings



Source: Authors (2018)

The active vertical loads (permanent and accidental) on the structure were defined following the prescriptions from ABNT NBR 6120:1980. The dimensions of slabs, beams, and columns were considered to define the permanent loads regarding the own weight of the structural elements. The non-structural permanent loads are from the own weight of smoothing, laying, lining mortar and the blocks as well as the thickness of the wall and its height. Table 2 shows the specific weights of the materials according to ABNT NBR 6120:1980.

Table 2: Specific weight of materials

Material	**Specific Weight (kN/m³)**
Reinforced Concrete	25
Cement and sand Mortar	21
Perforated Bricks	13

Source: ABNT NBR 6120 (adapted), 1980.

The coating load on the slab is composed of cement mortar and sand with 2 cm thickness, for regularization and laying, a granite floor of 1 cm thickness, totaling 0.7 kN/m², ceiling load of 0.5 kN/m². Thus, totaling 1.2 kN/m² of load on the slab.

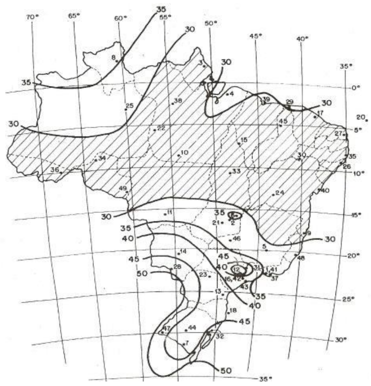
Wall loads were divided into walls into beams and walls on slabs. Both are made up of 15 cm thick perforated bricks. The walls on beams have a height of 2.60 m, resulting in a load of 4.68 kN/m, while the walls on the slabs have a height of 2.70 m, generating a load of 5.26 kN/m.

The variable actions, according to ABNT NBR 6120:1980, are those due to the use of the building, they are 1,5 kN/m² for bedrooms, breakfast room, kitchen, and bathroom; 2 kN/m² for the pantry, laundry area, and laundry; 3 kN/m² for stairs and corridors with public access and 2 kN/m² for areas without public access. Water tank loads and engine rooms were not taken into account for this study.

As stated earlier in this study, two ribbed slab plans were defined that include two apartments each, totaling an area of 203.72 m², which is the area of the slab in the pavement. Thus, a load of 1.5 kN/m² was adopted for the entire slab that contemplates the apartment, with the addition of a localized load of 0.5 kN/m², in the pantry, service area, and laundry.

According to ABNT NBR 6123:2013, variable wind actions were taken into consideration in the buildings. For this, it was necessary to define some parameters. The building is located in regions with the basic wind speed V0 of 30 m/s, according to the isometric map of figure 4.

Figure 4: Map of isopleth of basic wind speed



Source: ABNT NBR 6123 (2013)

For the topographic factor S1, which considers the variations of the relief, the terrain was adopted as flat or slightly hilly terrain and, thus the topographic factor was defined with the value S1=1.0.

The factor S2 takes into consideration the combination of the roughness effect of the terrain, variation of the wind speed with a height above the terrain, and the dimensions of the analyzed building. Factor S2 was defined as category III and class depending on the height of the building.

To determinate the statistical factor, the buildings in this study were classified as belonging to group 2 of item 5.4 of the code ABNT NBR 6123:2013, thus, the factor S3 is set 1.0.

The drag coefficients (Ca) were calculated for each pavement type using the abacus in figure 4 of ABNT NBR 6123:2013, taking into account the heights relative to each pavement, as well as the dimensions in plan.

3.3. THIRD STEP – DESIGN AND VERIFICATIONS OF STRUCTURAL ELEMENTS

Initially, for the design of structural elements, some parameters were defined: aggregate diameter as 19 mm; environmental aggressiveness class II; characteristic tensile strength of steel, f_{yk}, from 500 MPa to 600 MPa and characteristic compressive strength of concrete, f_{ck}, of 25, 30, 35 and 40 MPa.

The verification and design steps of the structural elements were performed according to ABNT NBR 6118:2014 using software for structural projects in reinforced concrete.

Based on the ultimate limit states, concrete checks and the design of the reinforcement were carried out. Based on service limit states, the horizontal and vertical displacements of the elements of the structure were verified, as well as the crack control.

For the columns, an attempt was made to optimize the cross-sections, through the relation of resistance to requesting moment (M_{rd}/M_{sd}), trying to make this relationship as close to 1.0.

Judging the γ_z coefficient as a criterion of relevance, the first and second-order effects were verified regarding the overall analysis of the structure.

The reinforcements obtained in the calculations of all structural elements, columns, beams, and slabs of all floors were edited to obtain the smallest steel area.

3.4. FOURTH STEP – QUANTIFICATION OF INPUTS

After completing the verifications of the ultimate and service limit states and the overall stability of the structure, the quantity of steel, concrete, and formworks was generated through the structural design software of reinforced concrete structures.

3.5. FIFTH STEP – TECHNICAL PARAMETERS

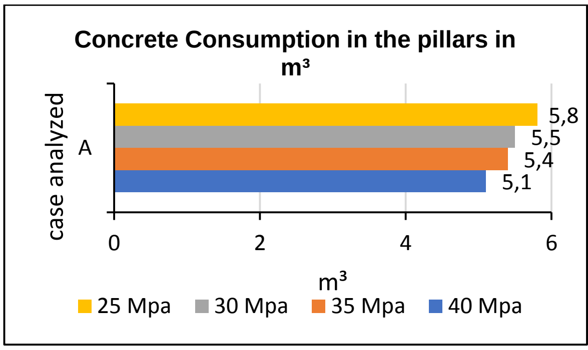
From the number of inputs, the technical parameters of each building were determined, taking into consideration the variations of pavements, as well as the variation in compressive strength of concrete. These studied parameters are steel consumption per cubic meter of concrete ($kg_{steel}/m^3_{concrete}$); consumption of formwork per meter cubic concrete ($m^2_{formwork}/m^3_{concrete}$) and average thickness of the typical floor.

4. RESULTS ANALYSIS

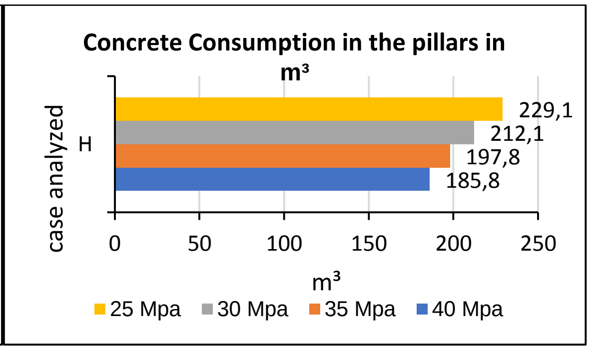
4.1. COLUMN ELEMENT

It was found that with the increase of the concrete f_{ck}, there was a reduction in the consumption of concrete. The average coefficient of variation of the eight buildings was 8.65%, with the case "A" presenting a variation coefficient of the concrete volume of 5.30%, according to figure 5.a). The case "H" presented a concrete volume variation coefficient of 9.06%, as shown in figure 5.b).

Figure 5: Concrete consumption in the pillars in the case – a) “A”; b) “H”



a)

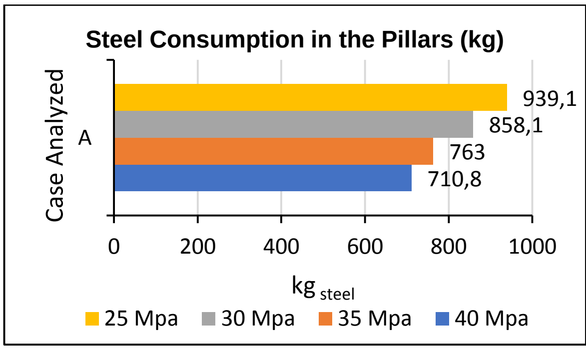


b)

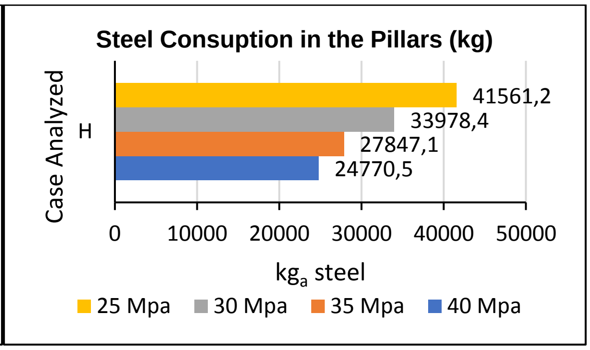
Source: Authors (2018)

Regarding the steel consumption of the columns, it was also observed that with the increase in the f_{ck} of concrete, this consumption was reduced. The average coefficient of variation was 16.30%. With case “A” presenting a variation coefficient of 12.39%, as shown in figure 6.a). While case "H" presented the highest variation coefficient, this being 23.14%, according to figure 6.b).

Figure 6: Steel consumption in the pillars in the case – a) "A"; b) "H"



a)



b)

Source: Authors (2018)

From the analysis of the consumption of concrete and steel in the columns, the relationship between steel consumption and concrete ($kg_{steel}/m^3_{concrete}$) was performed. As the rate of change in steel consumption was higher than the rate of change in concrete consumption, it resulted in the majority of cases resulting in a reduction in the $kg_{steel}/m^3_{concrete}$ ratio when the f_{ck} increased. However, in cases where this ratio increased when the f_{ck} was increased, this was because the rate of concrete change was higher than the rate of change in steel. Therefore, an average variation coefficient of 9.21% was observed, and table 3 shows the values obtained in all the proposed cases.

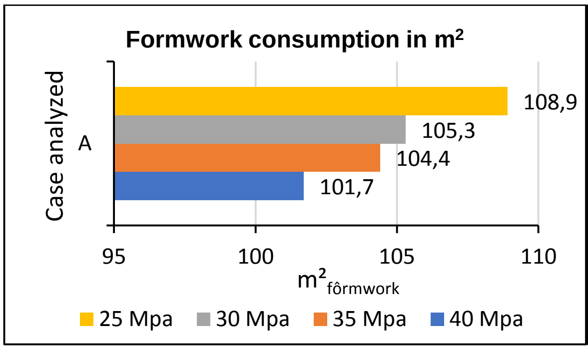
Table 3: Consumption Relation of the steel in the pillars (kg/m^3)

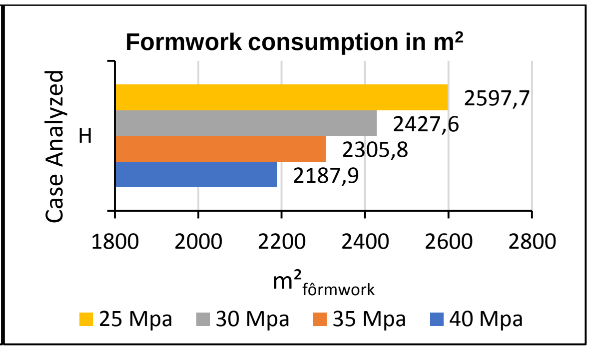
Case	25 MPa	30 MPa	35 MPa	40 MPa
A	161.91	156.02	141.30	139.37
B	200.05	227.82	195.73	196.78
C	208.73	222.44	228.28	215.27
D	259.14	241.27	235.12	221.58
E	261.63	219.94	212.62	181.64
F	219.44	206.93	189.30	172.20
G	174.26	150.44	143.05	148.58
H	181.41	160.20	140.78	133.32

It was observed that the $kg_{steel}/m^3_{concrete}$ ratio presented different behaviors according to the variation in the number of pavements type, and the highest buildings presented lower values in the $kg_{steel}/m^3_{concrete}$ ratio. Due to the high height of these buildings, they suffer more rigorously from the action of the wind, thus, there is greater difficulty to meet the requirements of global stability. In most cases, the minimum section of concrete necessary to resist the efforts, having a M_{rd}/M_{sd} ratio close to 1.0, was not able to meet the horizontal displacement limits. Therefore, it was necessary to increase the section of the columns, to meet the horizontal displacement limits. This made the $kg_{steel}/m^3_{concrete}$ ratio smaller when compared to the lower height cases.

As well as the consumption of concrete and steel, the consumption of formwork also decreased as the f_{ck} increased. An average coefficient of variation of 6.06% was obtained. Analyzing the “A” case, presented a variation coefficient of 2.83%, as shown in figure 7.a). While the “H” case, presented a greater coefficient of variation, this being 7.36%, according to figure 7.b).

Figure 7: Consumption of formwork of pillars in the case – a) “A”; b) “H”



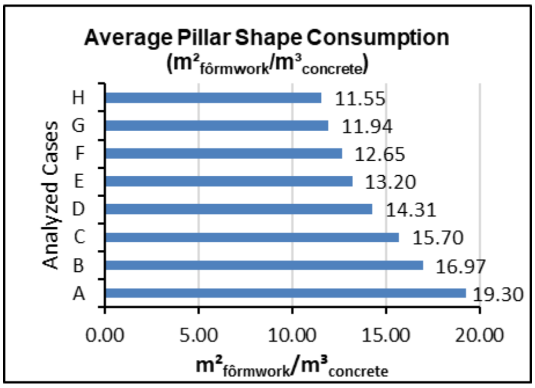


a) b)

Source: Authors (2018)

Analyzing the average consumption of formwork in each case concerning the volume of concrete $m^2_{formwork}/m^3_{concrete}$, it was observed that with the increase in the number of standard floors, there was a reduction in the relationship with a variation coefficient of 18.66%, as shown in figure 8. This is due to the fact previously explained in which the pillars of the tallest buildings presented higher consumption concrete than the lower buildings.

Figure 8: Average pillar shape consumption



Source: Authors (2018)

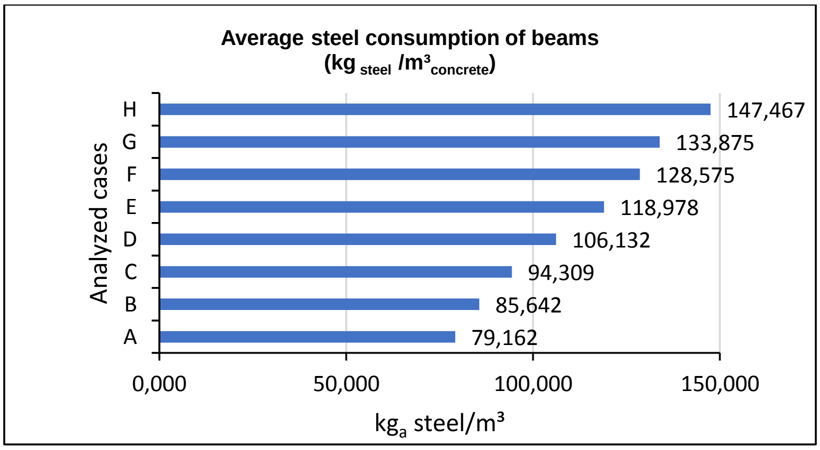
4.2. BEAM ELEMENT

The consumption of concrete was constant as the f_{ck} increased since the cross-sections of the beams were the same in all cases analyzed. Thus, the increase in consumption occurred only when the number of standard floors was increased.

Generally analyzing, the steel consumption in beams followed the behavior of slab consumption. In most cases, with the increase in f_{ck}, there was a small increase in steel consumption, with an average coefficient of variation of 0.94%.

It was also compared the average steel consumption of the beams of each case concerning the consumption of concrete, according to the increase in the number of floors. Based on this, it is concluded that with the increase in the number of floors, there was also an increase in the $kg_{steel}/m^3_{concrete}$ ratio of the beams, with a variation coefficient of 21.86%, as shown in figure 9.

Figure 9: Average steel consumption of beams



Source: Authors (2018)

The increase in the kg_{steel}/ $m^3_{concrete}$ ratio as it increased with the increase in the number of floors, occurred since the higher the height of the building, the greater the wind action on it. In this way, causing greater stresses to be generated, in this case, the beams end up having to absorb these greater stresses of moments, to guarantee the orthogonality of the structure, which causes the increase in the bending moment to increase the reinforcement.

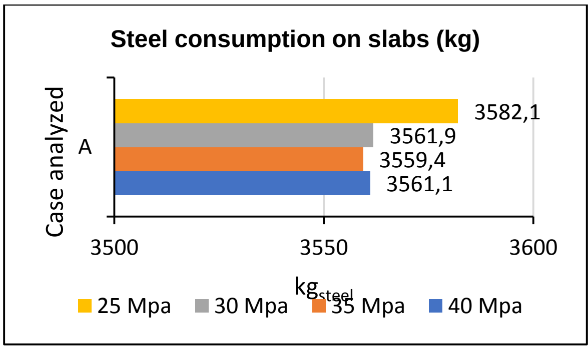
As well as the consumption of concrete, the consumption of formwork was constant as the f_{ck} increased, due to the same fact that the cross-sections were not changed. Thus, there was an increase in consumption only due to the increase in the number of standard floors. Making the relationship between the consumption of formwork and the concrete consumption of the beams, it was the same for the 32 cases analyzed.

4.3. SLAB ELEMENT

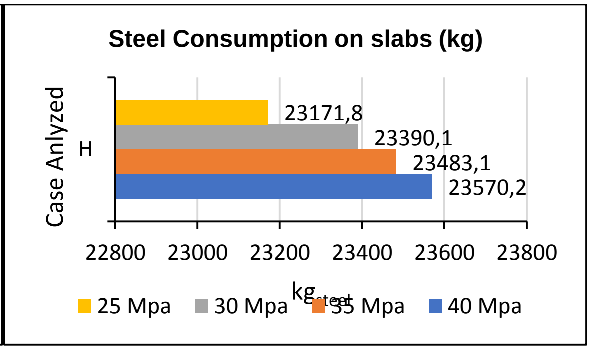
The concrete consumption of the slabs was constant when the f_{ck} of the concrete was increased, and this is because its dimensions, height of the concrete layer, and width of the rib were the same throughout all 32 analyzed cases. The increase in concrete on the slabs increased only when the number of standard floors was increased.

The steel consumption in the slabs in the majority achieved the same behavior. With the increase in f_{ck}, there was a small increase in steel consumption. However, this increase in consumption is around 0.79%. Where case "A" presented a variation coefficient of 0.30%, according to figure 10.a), and case "H", presented a variation coefficient of 0.73%, according to figure 10.b).

Figure 10: Steel Consumption in the slabs of case – a) “A”; b) “H”



a)

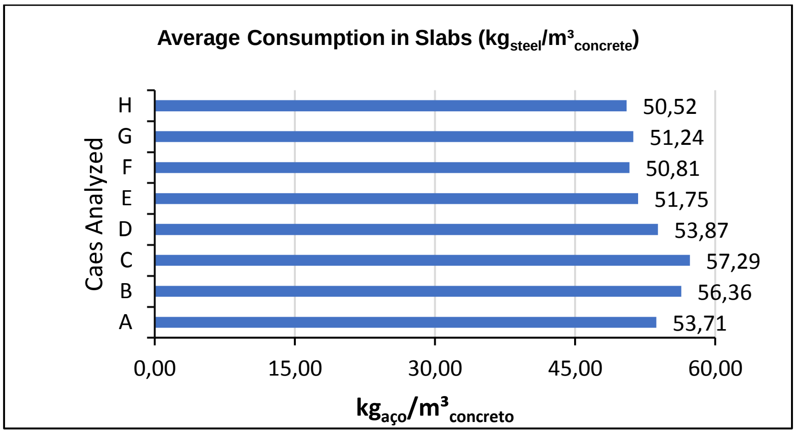


b)

Source: Authors (2018)

Comparing the average ratio of steel consumption to concrete consumption kg_{steel}/ $m^3_{concrete}$ of each case with the variation in the height of the number of floors, it was observed that the steel consumption did not present a great variation, with an average of 53.19 kg/m³ and variation coefficient of 4.83%, as shown in figure 11.

Figure 11: Average Consumption in slabs



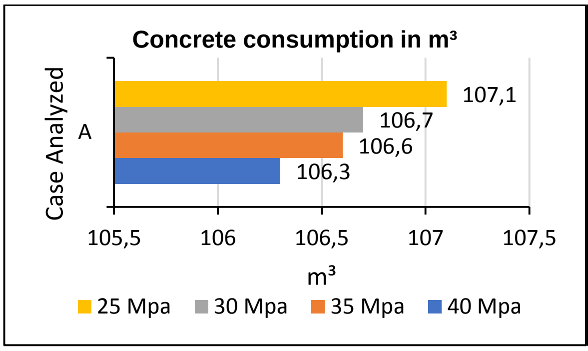
Source: Authors (2018)

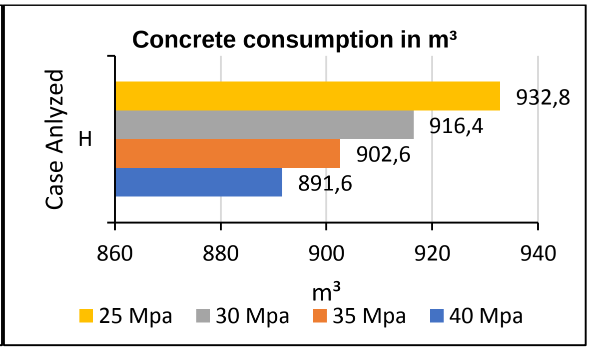
The slab form consumption was constant as the f_{ck} increased because the dimensions of the slabs have remained the same throughout the 32 cases analyzed. However, there was only an increase in the consumption of formwork with the increase in the number of standard floors. In this way, the relationship between the slab form consumption and the concrete consumption remained constant throughout all 32 cases.

4.4. GLOBAL ANALYSIS

When analyzing the overall concrete consumption of the structure concerning the increase in fck, it is possible to notice that there was a reduction in the consumption of this input, presenting an average variation coefficient of 1.12%. Also, it was observed that with the increase in the number of floors, there was also an increase in the variation of concrete consumption, with the taller buildings presenting higher coefficients of variation when compared to the lower buildings. Taking as an example the case “A” and the case “H”, which presented a variation coefficient of 0.31% and 1.95%, respectively, according to figures 12.a) and 12.b).

Figure 12: Concrete consumption in the case – a) "A"; b) "H"





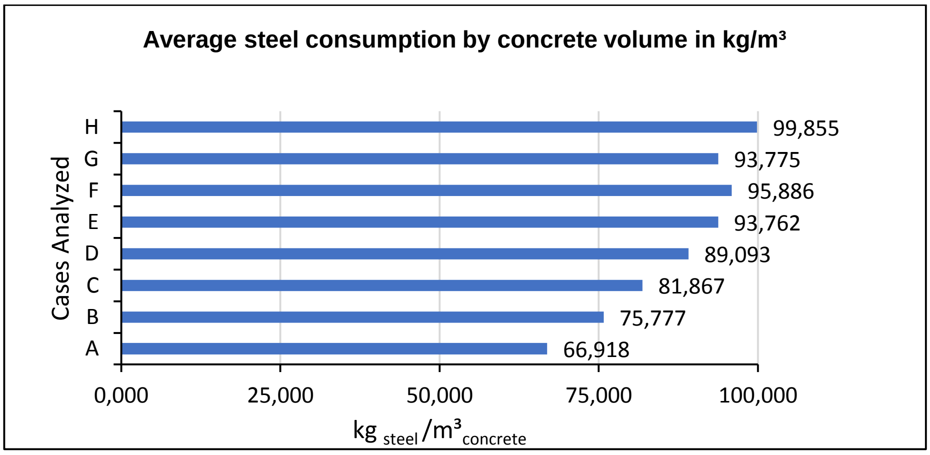
a) b)

Source: Authors (2018)

As well as the consumption of concrete, the consumption of global steel decreased as the f_{ck} increased, presenting a variation coefficient greater than that of concrete, which was 4.53%.

Analyzing the relationship between the average steel consumption and the average concrete consumption $kg_{steel}/\ m^3_{concrete}$ of each case, taking into account the variation in the number of floors, it was possible to observe that, as the number of standard floors increased, this relationship increased average steel/concrete consumption, with a coefficient of variation of 12.98%, as shown in figure 13.

Figure 13: Average steel consumption by the volume of concrete

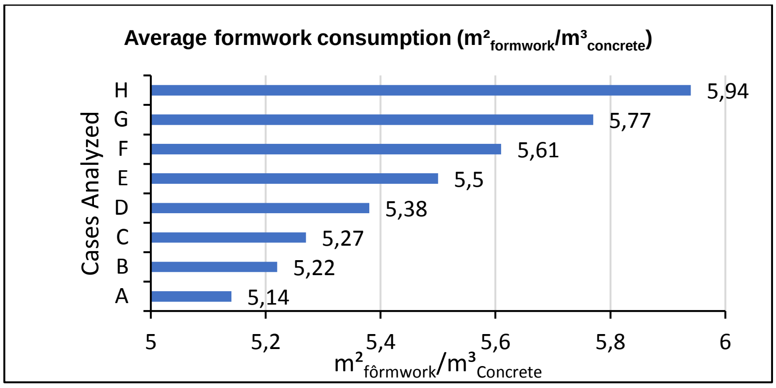


Source: Authors (2018)

It was observed that with the increase in f_{ck}, there was a reduction in the consumption of formwork whose analysis of its consumption was carried out through the relation of formwork consumption to the concrete consumption of the structure, presenting a variation coefficient of 0.78%.

Comparing the average ratio of the formwork consumption to the concrete consumption $m^2_{formwork}/m^3_{concrete}$ of each case, taking into account the variation in the number of floors, he observed that there was an increase in the formwork consumption with the increase in the number of floors, with a coefficient variation of 5.15%, according to figure 14.

Figure 14: Average formwork consumption

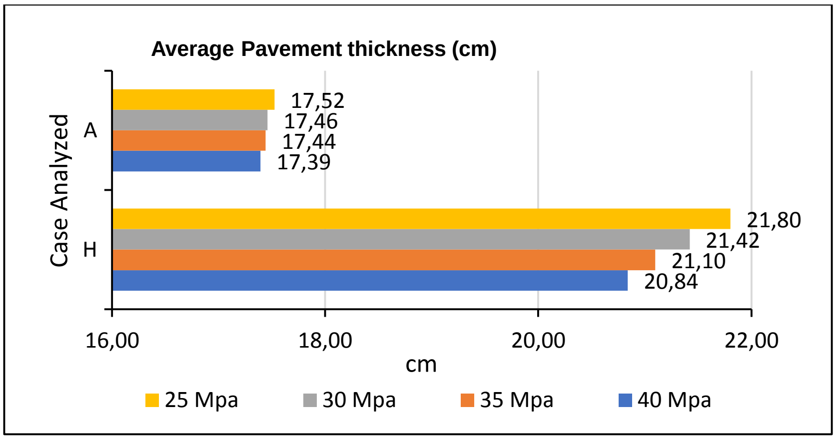


Source: Authors (2018)

The average thickness of the pavement is one of the most relevant indexes, as it is taken as a basis for estimating the volume of concrete in the structure. This parameter is a ratio of the volume of concrete to the total area of the pavement.

All cases in this study showed a reduction in the average pavement thickness concerning the increase in fck. Taking as an example the cases "A" and "H". In case "A" there was a small reduction in the average pavement thickness, presenting a variation coefficient of 0.31%. In the case of "H", it presented a greater reduction, with a variation coefficient of 1.95%, as shown in figure 15.

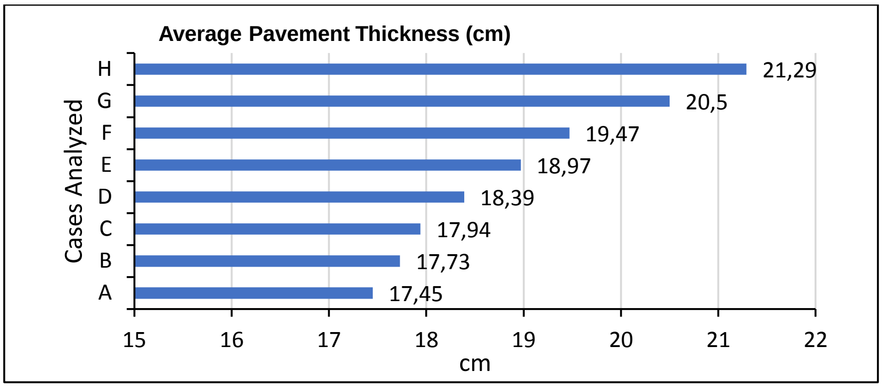
Figure 15: Average Thickness of the standard pavement in cases "A" and "H"



Source: Authors (2018)

Analyzing the average of the average thickness of each case, there was an increase in the average pavement thickness, as the number of standard pavements increased, with a variation coefficient of 7.24% as shown in figure 16.

Figure 16: Average thickness of the pavement type



Source: Authors (2018)

The increase in the average thickness, according to the increase in the number of pavements, occurs because the greater the height of the building, the greater the incidence of second-order effects on the structure.

5. CONCLUSION

Based on what was developed in this work, it was observed that the choice of concrete f_{ck} has great importance in the elaboration of the structural design since it is responsible for the structural behavior and consumption of the structure's inputs.

The increase in f_{ck} and the number of standard pavements did not result in significant variations in steel consumption in the slabs.

In beams, the increase in f_{ck} also did not show great variations in steel consumption. However, with the increase in the number of floors, there was a significant increase in steel consumption.

In the case of pillars, they showed a great reduction in steel consumption as the f_{ck} increased. However, the $kg_{steel}/\ m^3_{concrete}$ ratio showed a different behavior, that is, the tallest buildings presented a lower value of $kg_{steel}/\ m^3_{concrete}$ ratio when compared to those of lesser height.

As for the consumption of the work, with the increase in f_{ck}, there was no considerable reduction in the consumption of concrete in cases with a lower number of floors, when compared to cases with a higher number of floors. Taking the “A” and “H” cases as an example:

the “A” building, which had the lowest height, presented a variation coefficient of 1.63%; while the “H” building, which had the highest height, presented a coefficient of 6.12% variation.

Also, the shape consumption of beams and slabs continued steadily. On the other hand, there was a reduction in the shape consumption of the columns concerning the volume of concrete, as the number of floors increased.

This work provides data intending to assist the feasibility studies of similar projects in regions where the basic wind speed is 30 m/s with a geometric ratio of 1: 1, to serve as a subsidy for choosing the compressive strength that can guarantee the lowest cost. The design of multi-floor buildings has several peculiarities, making it impossible to generalize the quantifications, thus, it is suggested that there be further research on the subject addressed.

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