Acta Scientiarum

http://periodicos.uem.br/ojs
ISSN on-line: 1807-8664
Doi: 10.4025/actascitechnol.v46i1.64476

CIVIL ENGINEERING

Seismic behavior of large concrete panel prefabricated structures in relation to structural joints

Yamila Concepción Socarrás Cordoví*, Eduardo Rafael Álvarez Deulofeu, Ingrid Vidaud Quintana, Manuel Calzadilla Téllez, and Roberto Pompa Nuñez

Departamento de Ingeniería Civil, Universidad de Oriente, Cuba. *Author for correspondence. E-mail: ysocarrascordovi@gmail.com

ABSTRACT. There is evidence of good performance against large magnitude earthquakes, of the buildings built with the I-464 system in Armenia and Chile. This is a system made up of prefabricated panels and slabs, which are joined through wet joints; rigid at the level of the superstructure and articulated in a continuous foundation of cast-in-situ reinforced concrete. However, the possibility of the appearance of potential seismic damage in buildings built in Santiago of Cuba is not ruled out, due to the state of deterioration of the elements and joints, as well as the structural transformations carried out by the residents. That is why this research is designed as a purpose, to analyze the seismic behavior in relation to the structural joints. To meet this objective, 16 hypothetical variants are evaluated, which are prepared based on the statistical analysis of the diagnosis made to a sample of 200 buildings, as well as 4 variants that correspond to the original typologies. Multi-mass models are used for the dynamic analysis by SAP 2000 v20, where the joints between the precast elements were not explicitly modeled and were assumed to be rigid. The calculation requests in the joints were obtained through 442 'section cuts' in each of the five-level variants and 367 'section cuts' in the four-level variants. Then, both the resistance of the concrete to shearing and the resistance of the weld seams that join the protruding steel bars are evaluated, as well as checking the possibility of creep of these bars. It is concluded that only in the 5-level variants, where pathological damage is contemplated, there is a danger of failure of horizontal joints because the steel can reach the design stress and flow. Likewise, vertical joints can fail due to shear and due to the creep of the steel.

Keywords: large soviet panel; prefabricated system; joint check; deteriorated joints; seismic hazard.

Received on July 24, 2022.
Accepted on June 21, 2023.

Introduction

Prefabricated structures have suffered proportionally the same seismic damage as other structural systems and cannot be thought of as being intrinsically more vulnerable just because they are prefabricated. However, it is estimated that the deficiencies in terms of conception, execution and pathological damage that affect the joints cause a decrease in their rigidity and also an accelerated deterioration of these structures, as argued by Socarrás Cordoví and Vidaud Quintana (2017). The joints have a significant impact on their seismic behavior, becoming the neuralgic points of prefabricated structures.

Authors such as Holly and Abrahoim (2020), Gunawardena, Ngo, Mendis, and Kumar (2017), Zhou, Zhi, Fan, Jiao, and Qian (2020), Benjumea, Saiid, and Itani (2020), among others. Holly and Abrahoim (2020) state that the structural integrity of precast concrete structures depends mainly on the joints between the precast elements.

In prefabricated panel structures, Vaghei, Hejazi, Taheri, Jaafar, and Ali (2014), observed cracking along the panels and the connection of precast walls, during the application of an incremental lateral load. The Fema-154 (Federal Emergency Management Agency [Fema], 2015) report emphasizes that, in precast concrete structures based on large panels, due to seismic action, cracks appear in the horizontal and vertical joints, including spalling in the joints between the panels and the foundation. That is why this report recommends the execution of both horizontal and vertical, hyperstatic and humid joints.

But despite the damage evidenced, the structures based on large panels have shown adequate seismic behavior. In particular, there is evidence of good behavior in the face of large-magnitude earthquakes, of the buildings built with the I-464 system in Armenia and Chile. This is a system made up of prefabricated panels and slabs, which are joined through wet joints; rigid at superstructure level and articulated in a strip foundation of cast-in-situ reinforced concrete

In the province of Santiago of Cuba, the area of greatest seismic hazard in Cuba, there is a heritage built with this I-464 system, popularly known as the Great Soviet Panel. A total of 769 buildings were built in the province. Two types of buildings were implemented, with a balcony and without a balcony, basically with 4 or 5 levels.

In these buildings, with more than 50 years of operation, without a systematic conservation and maintenance policy, pathological damage has been detected in their structural elements and joints; as well as transformations in weight and rigidity, carried out by the inhabitants. Being probable, based on the investigations carried out in the territory, the appearance of potential seismic damage.

The studies that have been carried out range from the characterization of the materials in the current conditions of exploitation, the evaluation of the seismic vulnerability, to the prediction of the seismic behavior starting from the measurement of the periods of oscillation through the environmental vibrations (Tev). Socarrás Cordoví et al. (2020a) and Socarrás Cordoví, González Díaz, Álvarez Deulofeu, González Fernández, and Roca Fernández (2020b) obtained in the elements with pathological deterioration, a poor quality of the concrete and the compressive strength decreases by 25.78% in relation to that prescribed in the original project. Socarrás Cordoví, González Díaz, and Álvarez Deulofeu (2022) show, with respect to steel, that the reduction in the diameter of the corroded bars has an appreciable reduction (37.5%) in their yield stress.

In the article by Socarrás Cordoví, Álvarez Deulofeu, and Moreno Roche (2020c), the impact of some transformations in weight and stiffness on seismic behavior is appreciated. Likewise, Socarrás Cordoví, Álvarez Deulofeu, and Lora Alonso (2021a) observes that, in three instrumented buildings, the values of the periods according to the environmental vibrations (Tev), correspond to periods in the range of those expected before the design seismic action, due to the deterioration of the stiffness. At the same time, Socarrás Cordoví, Álvarez Deulofeu, and Lora Alonso (2021b) analytically corroborate the previous results. On the other hand, Socarrás Cordoví, Álvarez Deulofeu, and Galbán Rodriguez (2021c) assess the seismic behavior in relation to soil factors, the possibility of overturning being probable in these buildings. Also, like Socarrás Cordoví and Álvarez Deulofeu (2021) they evaluate the seismic vulnerability in deteriorated and transformed buildings, concluding that the prefabricated system does not meet all the current requirements of earthquake resistant design.

In order to continue analyzing the possibility of the appearance of potential seismic damage in buildings built in Santiago of Cuba with the Great Soviet Panel, this research aims to analyze the seismic behavior in relation to structural joints. To meet this objective, 16 hypothetical variants analyzed in Socarrás Cordoví, Álvarez Deulofeu, and Pupo Sintras (2021d), which are made based on the statistical analysis of the diagnosis made to a sample of 200 buildings, as well as 4 variants that correspond to the original typologies.

Multimass models are used for the dynamic analysis by SAP 2000 v20, where the joints between panel-panel, slab-slab and panel-slab elements were not explicitly modeled and were assumed to be rigid. The calculation requests in the joints were obtained through 442 'section cuts' in each of the five-level variants and 367 'section cuts' in the four-level variants. Then, both the resistance of the concrete to shearing and the resistance of the weld seams that join the protruding steel bars are evaluated, as well as checking the possibility of creep of these bars. It is concluded that only in the 5-level variants, where pathological damage is contemplated, there is a danger of failure of horizontal joints because the steel can reach the design stress and flow. Likewise, vertical joints can fail due to shear and due to the creep of the steel.

Material and methods

The research was structured in three stages as explained below:

Stage I: A search for documentary information was carried out on the type of joints of the Great Soviet Panel prefabricated system in terms of dimensions of the joints and their execution, quality of the concrete, as well as quality and diameter of the steel bars in the structural joints.

Stage II: The stresses acting on the structural joints between slabs, between slabs and panels and between panels are determined. For this, 20 structural models are conceived. Of which, 4 models correspond to the original typologies. The rest are hypothetical models, based on the statistical result of the diagnosis made.

A multimass model is used for the dynamic analysis by SAP 2000 v20, based on the properties of the materials, the geometry and the links between component elements. The panels are modeled like the slabs, as finite 'shell' elements, continuously joined together to produce a rigid and homogeneous structural system. Likewise, the stair slabs are modeled as finite elements type 'Shell' connected to the panels and slabs. The joints between panel-panel, slab-slab and panel-slab elements were not explicitly modeled and were assumed to be rigid. The plinths are considered to be simply supported on the continuous reinforced concrete base. Figure 1 shows the isometrics of the geometric models of the original project.

Four hypothetical variants are considered for each typology. Table 1 shows the dimensions of the buildings in plan and elevation for each type of building (1, 2, 3, 4A) and below the description of each of the hypothetical variants by typology.

Description of the hypothetical variants:

Variants 1, 2, 3, 4B: Water tanks in the patios, masonry wall in the multipurpose areas. Filling of the lattices panels E-5 or E-4. Closing of the opening of the main door in a corner apartment, with the opening of panel E-6, in the typology without balcony.

Variants 1, 2, 3, 4C: An opening of 0.90 x 2.35 m to a panel (I-8 or I-10) of the 2nd level, plus Subgroup B. Opening of 1 x 1 m in slab P-7.

Variants 1, 2, 3, 4D: A 0.90 x 2.35 m opening to a 1st level I-7 panel, and addition of a masonry wall plus Subgroup B. 1 x 1 m opening in slab P-7.

Variants 1, 2, 3, 4E: The most critical variant among subgroups B, C, D, plus pathological damage to elements in the kitchen-bathroom and patio areas.

The resistances of the materials are obtained from destructive and non-destructive tests on concrete and steel. The research by Socarrás Cordoví et al. (2020a) and (2020b) details the results of the concrete tests that were carried out both on elements that are still in the warehouse area of the precast plant and on elements that make up the buildings. Likewise, in the study of Socarrás Cordoví et al. (2022) the properties of steel are specified. See Table 2.

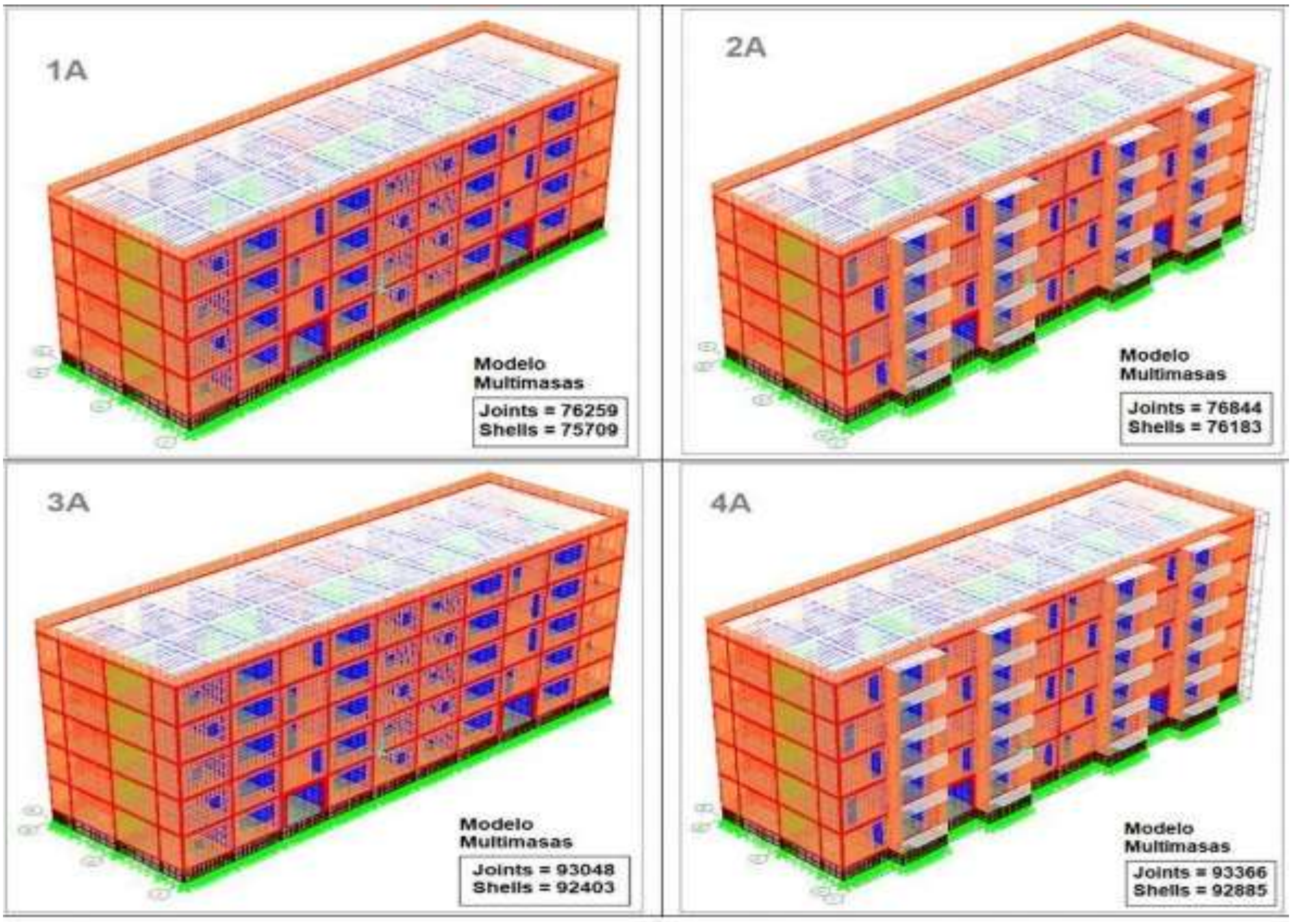


Figure 1. Structural models of the typologies according to the original project. SAP 2000v20.

Table 1. Nomenclature and sizing of the variants analyzed.

Typologies	Group 1	Group 2	Group 3	Group 4
Original Project	1A	2A	3A	4A
Building length (L), mm	32000	32000	32000	32000
Building width (A), mm	9600	9600	9600	9600

Group 1: 4 floors, without balcony; Group 2: 4 floors, with balcony; Group 3: 5 floors, without balcony; Group 4: 5 floors, with balcony.

The modulus of elasticity of precast concrete is calculated using the expression recommended by ACI 318R-19 (American Concrete Institute [ACI], 2019), but with a reduction greater than 40% as recommended by Lewicki (1968) for precast concrete buildings, since it is considered that the greatest deformation of structures with precast elements is due to the presence of joints. On the other hand, it is increased by 20% because the seismic action is of short duration, totaling a penalty of 28%. The shear modulus G is obtained from the modulus of elasticity E, assuming for the concrete a Poisson's ratio ν = 0.17.

Table 2. Characteristics of the materials.

Material characteristics			
Steel	Diameters (mm)	fyk in elements without pathological damage (MPa)	fyk in elements with pathological damage (MPa)
Corrugated	9.5	328.72	205.45
	12	324.43	202.76
Smooth	3	948.58	592.86
	6	397.40	248.37
	8	554.62	346.63
	f´ck (MPa)	Modulus of deformation E (MPa)	
Precast concrete in elements without pathological damage	16.00	13536.00	
Precast concrete in elements with pathological damage	12.79	12102.23	
Lattice Filled Concrete	10.00	10701.14	

The use of linear analysis requires the introduction of stiffness modifiers to reflect the degree of cracking and inelastic action that occurs in the elements immediately before yielding, according to Fema-273 (Federal Emergency Management Agency [Fema], 1997) and ACI:318-19; also, to visualize in the structural models the incidence of the openings of the elements and the pathological damages present. The modifiers are obtained iteratively, through the calibration of the structural models, until the fundamental period of the model is greater than the natural or empirical one (T model > T natural or T empirical), since with the increase in the levels of movement the natural periods increase. In this sense Chopra (2014) argues that the natural period of vibration of the linear system is equal to the period of the elasto-plastic system only under oscillations with small amplitudes.

In the calibration of the models of subgroup E, since they are buildings with large prefabricated panels, the contributions of Polyakov (1974) and Chopra (2014) are evaluated and values of the environmental vibration periods (Tev) are assumed from 0.14 to 0.24 s, according to the results of Socarrás Cordoví et al. (2021a). Then, in the iterative calibration process, we start from the modifiers used in the preceding subgroups, and from the criterion that the fundamental period of the models be greater (between 2-15%) than the period of the vibrations environmental [T generated model > (1.02 ~1.15) Tev]. Table 3 summarizes the bending stiffness modifiers used in the calibrated structural models.

Table 3. Stiffness modifiers used in the different structural models.

Variant	Stiffness modifiers								
	Plinths	ILP	ELP	ICP	ECP	PDP	Slabs	PDS	LF
A	0.70 EI	0.70 EI	0.70 EI	0.70 EI	0.70 EI	-	0.50 EI	-	-
B	0.70 EI	0.70 EI	0.70 EI	0.70 EI	0.70 EI	-	0.50 EI	-	0.15 EI
C	0.70 EI	0.35 EI	0.70 EI	0.70 EI	0.70 EI	-	0.50 EI	-	0.15 EI
D	0.70 EI	0.70 EI	0.70 EI	0.35 EI	0.70 EI	-	0.50 EI	-	0.15 EI
E	0.70 EI	0.35 EI	0.35 EI	0.35 EI	0.35 EI	0.15 EI	0.25 EI	0.10 EI	0.15 EI

ILP: Interior Longitudinal Panels; ELP: Exterior Longitudinal Panels; ICP: Interior Cross Panels; ECP: Exterior Cross Panels; PDP: Pathological Damaged Panels; PDS: Pathological Damaged Slabs; and, LF: Lattice Fills.

The permanent (G) and use (Q) loads are defined according to the NC 283:2003 (*Oficina Nacional de Normalización* [NC], 2003a) and NC 284:2003 (*Oficina Nacional de Normalización* [NC], 2003b) standards, respecting the considerations of the original projects. See Table 4.

The additional considerations in the invariants of the structural modeling regarding the modeling of the loads are: water tanks with a height of 1.20 m in the service patios; 15 cm thick block walls that were added in the multipurpose areas and the lattice filling with 10 MPa concrete. The own weight of all the elements is generated by the SAP2000v20 software from the specific weight of the material specified by NC 283:2003 (25 kN m^{-3}) for prefabricated elements.

Table 4. Permanent loads and use according to current Cuban regulations.

Permanent Loads			
Roof	Three layers of gravel asphalt		0.280 kN m^{-2}
mezzanine	Filled	0.180 kN m^{-2} cm	1.955 kN m-2
	Mortar	20.00 kN m^{-3}	
	Mosaic	0.230 kN m^{-2} cm	
Ladder			1.960 kN m^{-2}
Utilization Loads			
Roof	Cover Drain per gutter		2.000 kN m-2
mezzanine	Rooms of common houses		1.500 kN m^{-2}

Seismic (S) loads are modeled according to NC 46:2017 (*Oficina Nacional de Normalización* [NC], 2017), with the Response Spectrum Method (RSM) and the Static Equivalent Method (SEM), for the latter the fundamental periods of the modal analysis are considered in the three main directions of the actions seismic (both horizontal and vertical). 100% of the seismic load in one of the main directions is combined, simultaneously with 30% in the remaining directions. In the case of the SEM the seismic load, according to the NC 46:2017 standard, in the vertical direction is modeled as an increase in the total permanent load that includes the structure's own weight, estimated as 20% of the permanent load by the response acceleration for a short period determined in the Design Spectrum for the soil profile considered.

Also, in each of the floors, the accidental eccentricities of the centers of mass with respect to the centers of rigidity are considered. For the proposed model, it is verified that the centers of rigidity of each of the floors approximately coincide with their centers of mass, so their position was assumed to be the same for all the floors. In the RSM method, the CQC was used as the modal superposition formula, which considers the proximity of the modes in the response, through the modal correlation coefficients. In addition, it is verified that the sum of the modal contribution factors for each of the main directions of the seismic action is close to unity, as proposed by Chopra (2014).

The calculation response spectrum corresponds to the design earthquake specified according to NC 46:2017 for residential buildings and is obtained for the seismic zone under analysis, taking into account the physical-mechanical characteristics of the soil profile where they are located Great Soviet Panel residential buildings studied. Additionally, the reductions of the spectral ordinates for the energy dissipation of the Great Soviet Panel system are considered, assuming a natural ductility. The considerations for the elaboration of the spectrum shown in Figure 2 are detailed below:

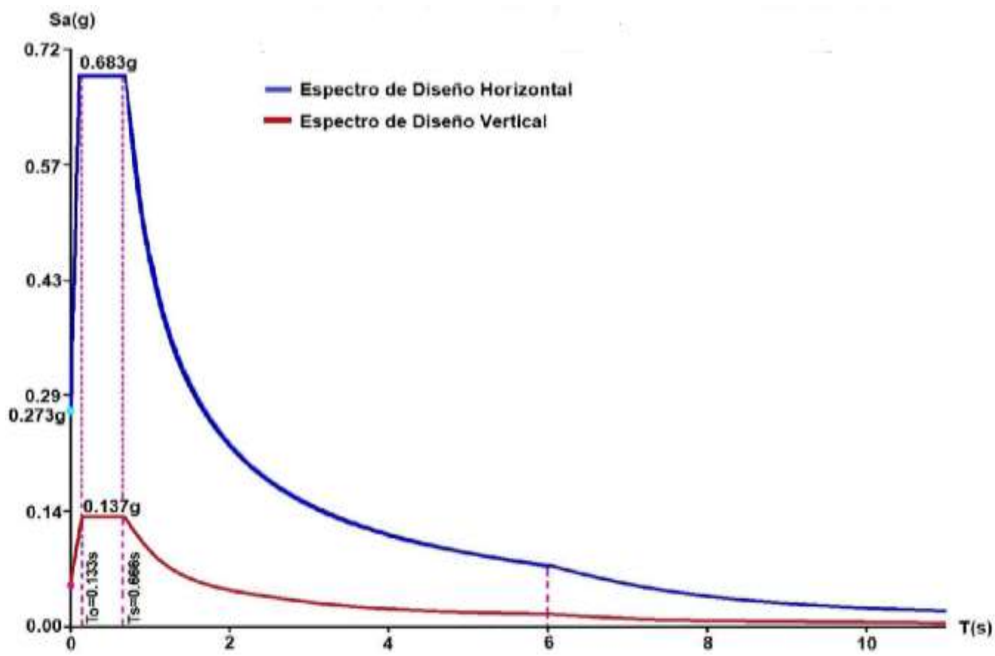


Figure 2. Design spectrum for horizontal and vertical loads according to NC 46:2017.

O Very high seismic hazard zone (5), where the maximum horizontal ground accelerations (0.3 g) for the design earthquake are due not only to the seismic zone but also to the category of the work. In the case of residential buildings, classified as 'Ordinary', a 'Basic Earthquake' is recommended, which for periods of useful life of 50 years and an accepted probability of exceedance of 10% according to a level of seismic protection D, correspond to a return period of 475 years from the design earthquake.

O Type of soil: profile D, associated with rigid soils of any thickness that meets the shear wave velocity criterion (180 m s^{-1} ≤ Vs ≤ 360 m s^{-1}), or rigid soil profiles of any thickness that satisfy either of the two conditions shown.

1) 15 ≤ N ≤ 50, N: Average number of blows of the standard penetration test [blows/foot];

2) 50 kPa ≤ Su ≤ 100 kPa Su: Average shear strength of the undrained test in cohesive soil strata.

O Structural system: E2 (Wall system):

O Ductility reduction factor R = 1.5, because it is valued that they are prefabricated structures designed by repealed codes, with little ductility of the steel of the structural elements and inadequate detailing of the sections of the elements. This criterion was handled by assessing the contributions of the COVENIN 1756:2001 (Consejo Superior de Fondonorma [COVENIN], 2001). Said standard assigns the value of the response reduction factor R = 1, for buildings made up of prefabricated members up to the year 1967.

The loads combinations used are:

Combo 1: 1.2G + 0.25Q + 1.0Sx + 0.3Sy + 0.3Sz
Combo 2: 1.2G + 0.25Q + 0.3Sx + 1.0Sy + 0.3Sz
Combo 3: 1.2G + 0.25Q + 0.3Sx + 0.3Sy + 1.0Sz
Combo 4: 0.9G + 1.0Sx + 0.3Sy + 0.3Sz
Combo 5: 0.9G + 0.3Sx + 1.0Sy + 0.3Sz
Combo 6: 0.9G + 0.3Sx + 0.3Sy + 1.0Sz
Combo 7: 1.2G + 1.6Q

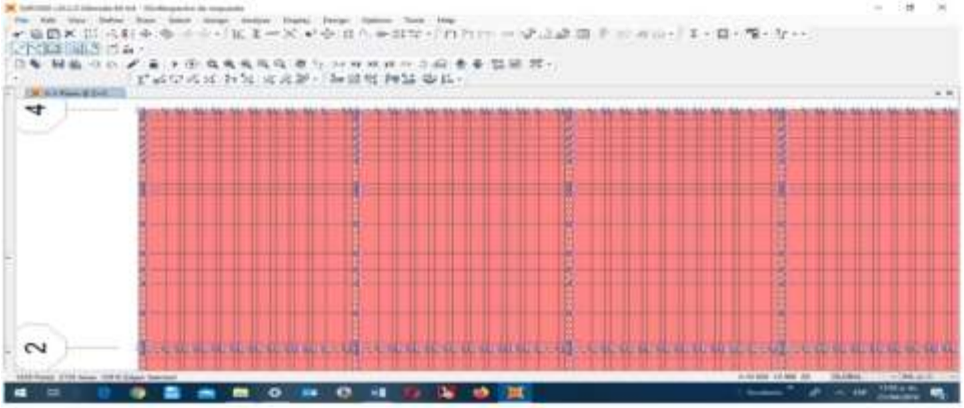
They are implemented in the 'section cuts' structural models, which allow obtaining the calculation stresses in the areas of the joints between slabs, between slabs and panels and between panels. The calculation requests in these joints were obtained through 442 'section cuts', in each of the variants of five levels and 367 'section cuts' in those of four levels. See Figure 3.

Stage III: The structural joints between slabs, between slabs and panels and between panels are checked, through the expressions that are summarized in Table 5. The contributions of Lewicki (1968) and Baykov and Sigalov (1980) are valued in terms of to the checking of the concrete shear strength, of the Fema-310 report (Federal Emergency Management Agency [Fema], 1998) that suggests the checking of the weld fracture and of the American Concrete Institute regulation (ACI-318-19), in relation to the restriction of the creep of reinforcement at connections. Table 5 shows a summary of the formulations used to check the limit states mentioned above.

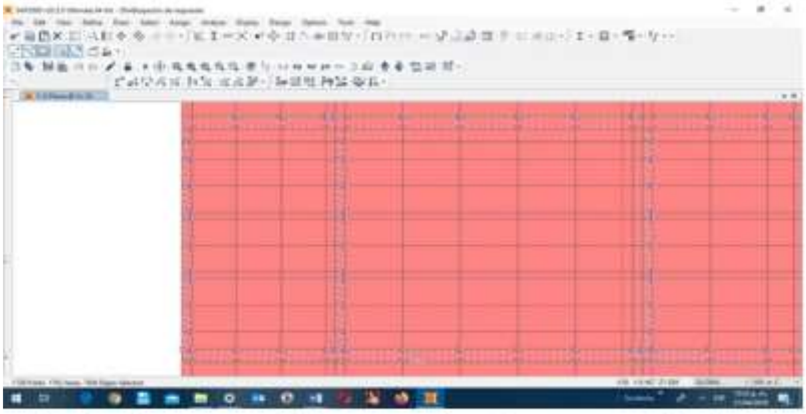
Where:

δch, Lch, hch: depth, length, height of the key (mm);
nch: Number of keys (dimensionless);
f´c: Compressive strength of the concrete of the joint (MPa or N mm^{-2});
Rtr: Tensile strength of the concrete in the joint. (MPa or N mm^{-2});
Tu: Shear force that the joint can transmit. (N);
RƬ: Concrete resistance to shearing, it is recommended: RƬ = 0.15f'c, where f'c is the resistance of the concrete to compression of the joint. (MPa or N mm^{-2});
Bj: Useful shear section (mm^2);
Nc: compression force of permanent action (N);
Nu: Axial force supported by the weld beads (N);
∑ Ls: Sum of the weld beads (mm);
FExx: Electrode resistance (MPa or N mm^{-2});
hs: thickness of the weld bead (mm);
fy: Yield strength of the steel (MPa or N mm^{-2});
β: Coefficient that depends on the form of welding (dimensionless);
Aϕ: Area of a bar (mm^2).

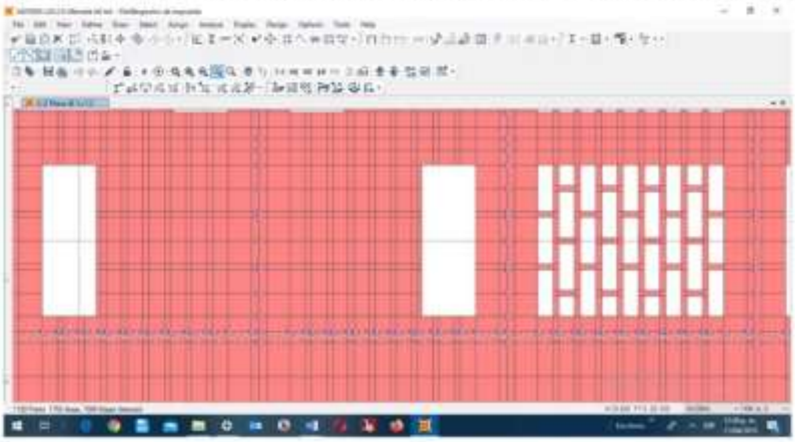
In damaged joints, the expressions of Coronelli and Gambarova (2004) and Du, Clark, and Chan (2005), were used to define the strength of concrete and steel.



A) Section Cuts between slabs



B) Section Cuts between slabs and panels and between panels in the transverse direction.



C) Section Cuts between slabs and panels and between panels in the longitudinal direction.

Figure 3. Section cuts in the structural models.

Table 5. Expressions for checking structural joints.

Conditions to check	Expressions	Source
Shear check of horizontal joints between slabs and between slabs and panels, as well as vertical joints between panels.	$Tu \leqslant (\delta ch)\ (Lch)\ (nch)\ (f'c)$ $Tu \leqslant 2\ (hch)\ (Lch)\ (nch)\ (Rtr)$ o $Tu \leqslant 2\ (hch)\ (Lch)\ (nch)\ (Rtr) - 0,7\ Nc$*	Baykov and Sigalov (1980)
Shear strength check at vertical joints between panels.	$Tu \leqslant Bj\ R\tau$ **	Lewicki (1968)
Welding resistance check	$Nu \leqslant \sum Ls$ R welding design A welding R welding design = $_{\varphi}$ Fw $Fw = 0,6\ FExx$ A welding = $_{\beta}$ hs x 1 cm	McCormac (2008)
Checking the possibility of creep of steel bars	Creep resistance = fy Aϕ # bars	

Note: *This expression is used when there is a permanent action compressive stress (N). **Only for checking shear strength at vertical joints between panels.

Results and discussion

The files of Emproy-15, responsible for the project of these buildings, provided the necessary documentation for the definition of the type of joints between the structural elements. Subsequently, the type of joints was corroborated through the inspection of some prefabricated elements in existence in the storage area of the ‘Gran Panel Santiago’ prefabricated plant. See Figure 4. Table 6 shows the dimensions of the trunks. The compressive strength of the concrete in the joints (wet type) reaches 20 MPa, the welding between the steel bars is manual with 483 MPa resistance electrodes. The protruding steel bars have a diameter of 13 mm and yield strength of 300 MPa and corroborated through destructive tests by Socarrás Cordoví et al. (2022).

With the calculation requests (bending, shear and axial moment) that appear in the joints, offered by the ‘section cuts’ introduced to the model, and obtained for each of the load combinations specified in NC 46:2017, it is carried out their structural check. Thus, the shear strength, weld strength, and yield stress of the steel are checked. Finally, the safety factors are calculated in each case (FS). The result is summarized in Table 7 and 8.

Figure 4. Borders of prefabricated elements of the Great Soviet Panel. Source: Authors (2022).

Table 6. Values of parameters used to check the joints.

Horizontal joints between slabs						
δch (mm)	Lch (mm)	hch (mm)	nch	Ls (mm)	hs (mm)	# bars
80	120	350	3	80	8	6
Horizontal joints between slabs and panels						
150	340	380	2	80	8	4
Vertical joints between panels						
220	120 o150	410	3	120	8	6

Table 7. Check of structural joints. Variants 1 and 2.

Joints	Total	Variant	1A			1B			1C			1D			1E		
		Check	I*	II*	III*	I*	II*	III*	I*	II*	III*	I*	II*	III*	I*	II*	III*
Horizontal Longitudinal Slabs- slabs	110	Resist	110	110	110	110	110	110	110	110	110	110	110	110	110	110	110
		FS >	5.4	15.6	5.4	5.3	15.5	5.5	5.3	15.3	5.4	5.3	15.5	5.4	5.2	15.0	3.4
Horizontal Transverse Slabs- slabs	150	Resist	150	150	150	150	150	150	150	150	150	150	150	150	150	150	150
		FS >	8.0	7.3	2.6	8.0	7.2	2.3	7.9	7.2	2.3	7.9	7.2	2.3	7.2	6.2	1.4
Horizontal Longitudinal Slabs -Panel	110	Resist	110	110	110	110	110	110	110	110	110	110	110	110	110	110	110
		FS >	16.5	13.5	6.3	16.4	13.2	4.5	16.3	13.2	4.4	16.3	13.2	4.4	15.0	10.8	3.1
Horizontal Transverse Slabs -Panel	150	Resist.	150	150	150	150	150	150	150	150	150	150	150	150	150	150	150
		FS >	24.9	12.5	5.8	24.7	9.8	4.6	24.7	9.8	4.6	24.7	9.8	4.6	25.8	8.5	2.5
Vertical Longitudinal Panel-Panel	180	Resist	180	180	180	180	180	180	180	180	180	180	180	180	180	180	180
		FS >	2.08	7.28	1.70	2.03	7.08	1.63	2.01	7.13	1.52	2.03	7.13	1.53	1.90	6.52	1.68
Vertical Transversal Panel-Panel	185	Resist	185	185	185	185	185	185	185	185	185	185	185	185	185	185	185
		FS >	2.7	15.6	3.6	2.2	15.0	3.6	2.0	15.0	3.4	2.0	15.0	3.5	2.0	14.3	2.7
			2A			2B			2C			2D			2E		
Horizontal Longitudinal Slabs- slabs	146	Resist	146	146	146	146	146	146	146	146	146	146	146	146	146	146	146
		FS >	5.1	15.0	4.8	5.1	14.8	4.8	5.0	14.7	4.8	5.0	14.7	4.8	4.8	14.4	3.1
Horizontal Transverse Slabs- slabs	150	Resist	150	150	150	150	150	150	150	150	150	150	150	150	150	150	150
		FS >	8.9	9.0	3.2	8.7	9.6	3.1	8.6	9.4	3.1	8.6	9.6	3.1	8.4	5.9	1.3
Horizontal Longitudinal Slabs -Panel	110	Resist	110	110	110	110	110	110	110	110	110	110	110	110	110	110	110
		FS >	16.4	13.1	6.1	16.2	13.0	6.0	16.0	13.0	6.0	16.1	13.0	6.0	14.8	10.4	3.1
Horizontal Transversal Slabs -Panel	150	Resist	150	150	150	150	150	150	150	150	150	150	150	150	150	150	150
		FS >	24.3	11.2	5.2	24.1	9.4	4.4	24.0	9.1	4.3	24.2	9.1	4.3	22.9	7.4	2.2
Vertical Longitudinal Panel-Panel	180	Resist	180	180	180	180	180	180	180	180	180	180	180	180	180	180	180
		FS >	1.6	6.5	1.5	1.6	6.4	1.4	1.6	6.3	1.4	1.6	6.4	1.4	1.5	5.3	1.1
Vertical Transverse Panel-Panel	185	Resist	185	185	185	185	185	185	185	185	185	185	185	185	185	185	185
		FS >	1.7	18.5	4.3	1.6	16.0	3.7	1.6	16.0	3.7	1.6	16.0	3.7	1.6	15.6	2.3

Table 8. Check of structural joints. Variants 3 and 4.

Joints	Total	Variant	3A			3B			3C			3D			3E		
		Check	I*	II*	III*	I*	II*	III*	I*	II*	III*	I*	II*	III*	I*	II*	III*
Horizontal Longitudinal Slabs- slabs	180	Resist	132	132	132	132	132	132	132	132	132	132	132	132	132	132	132
		FS >	4.8	13.3	5.2	4.8	13.0	5.2	3.9	13.0	5.2	4.8	13.2	5.2	4.3	13.0	1.6
Horizontal Transverse Slabs- slabs	132	Resist	180	180	180	180	180	180	180	180	180	180	180	180	180	180	170
		FS >	5.7	6.3	2.5	5.7	6.3	2.5	4.7	6.3	2.5	4.8	6.3	2.5	4.6	6.3	1.0
Horizontal Longitudinal Slabs -Panel	180	Resist	132	132	132	132	132	132	132	132	132	132	132	132	132	132	122
		FS >	14.8	4.2	1.7	14.7	4.2	1.7	14.7	4.2	1.7	13.9	4.2	1.7	8.8	4.1	1.0
Horizontal Transverse Slabs -Panel	216	Resist	180	180	180	180	180	180	180	180	180	180	180	180	180	180	140
		FS >	18.0	4.9	1.9	18.0	4.9	1.9	17.5	4.9	1.9	17.6	4.9	1.9	10.9	4.9	1.0
Vertical Longitudinal Panel-Panel	222	Resist	216	216	216	216	216	216	216	216	216	216	216	216	213	216	198
		FS >	1.3	4.6	1.2	1.3	4.6	1.2	1.3	4.5	1.2	1.3	4.6	1.2	1.0	1.0	1.0
Vertical Transversal Panel-Panel	172	Resist	222	222	222	222	222	222	222	222	222	222	222	222	222	222	220
		FS >	1.6	11.7	3.1	1.6	11.6	3.0	1.6	11.6	3.0	1.6	11.6	3.0	1.2	10.0	1.1
			4A			4B			4C			4D			4E		
Horizontal Longitudinal Slabs- slabs	180	Resist	172	172	172	172	172	172	172	172	172	172	172	172	172	172	172
		FS >	4.5	11.0	4.3	4.3	10.6	4.3	4.2	10.6	4.3	4.2	10.6	4.3	3.7	9.8	1.2
Horizontal Transverse Slabs- slabs	132	Resist	180	180	180	180	180	180	180	180	180	180	180	180	180	180	170
		FS >	6.3	5.3	2.1	6.2	5.3	2.1	6.2	5.3	2.1	6.2	5.3	2.1	5.9	5.0	1.0
Horizontal Longitudinal Slabs -Panel	180	Resist	132	132	132	132	132	132	132	132	132	132	132	132	132	132	119
		FS >	11.0	3.8	1.5	10.9	3.3	1.3	10.9	3.3	1.3	10.9	3.3	1.3	10.1	2.5	1.0
Horizontal Transversal Slabs -Panel	216	Resist	180	180	180	180	180	180	180	180	180	180	180	180	180	180	138
		FS >	13.6	4.9	1.9	13.5	4.1	1.6	13.4	4.1	1.6	13.4	4.1	1.6	12.1	3.1	1.0
Vertical Longitudinal Panel-Panel	222	Resist	216	216	216	216	216	216	216	216	216	216	216	216	213	216	195
		FS >	1.3	4.6	1.1	1.3	4.6	1.0	1.0	4.5	1.0	1.0	4.6	1.0	1.0	3.6	1.0
Vertical Transverse Panel-Panel	132	Resist	222	222	222	222	222	222	222	222	222	222	222	222	222	222	220
		FS >	1.4	11.6	3.0	1.3	11.6	3.0	1.3	11.5	3.0	1.3	11.6	3.0	1.3	8.8	1.1

*Note: I) Shear check; II) Weld check; III) Check the yield strength of the steel.

The horizontal joints between slabs and between slabs and panel, which are made at the story levels, essentially transfer the gravitational loads and the horizontal actions of the floor diaphragm. The vertical joints between panels fundamentally transfer the shear, guaranteeing a minimum displacement of the panels.

Taking into account the arguments of Holly and Abrahoim (2020), together with the evidence of adequate seismic behavior of buildings built with large panels joined through wet joints, it can be stated that the joints of the Great Soviet Panel prefabricated system are suitable for ensure good seismic performance. In particular, Holly and Abrahoim (2020) show vertical joints between panels, horizontal joints between panels and slabs, which are currently used and are suitable for seismic zones, which are very similar to those designed for the Great Soviet Panel precast system.

Figure 4 shows that the edges of the panels and slabs have grooves, teeth, indentations, pockets and protruding steel bars, which guarantee the execution of the wet joints. The grooves, teeth, crevices and pockets allow the creation of shear keys that act as mechanical locks when the elements are deformed. Vaghei et al. (2014) specifies that the most important thing is the resistance of the concrete with which the shear key is filled and not the configuration and quantity of these. The steel bars serve as structural links and provide additional strength to prevent elements from separating.

When the results of Table 7 and 8 are evaluated, it is observed that only in the 5-level variants (3E and 4E), where pathological damage is contemplated, there is a danger of joint failure. In the horizontal ones, the design yield stress of the joint steel is reached, which does not comply with the hypothesis assumed by the authors of not accepting energy dissipation in the joints, preserving their elastic behavior. The latter corresponds to the low ductility reduction factor assumed for the Great Soviet Panel buildings studied. In vertical joints, the same thing happens as for horizontal joints, together with shear failure, which is a brittle failure, unwanted in the face of seismic actions. Likewise, it is perceived that the safety factors in horizontal joints are greater than in vertical ones. This agrees with the results of cited by Vaghei et al. (2014), where the horizontal joints are more rigid than the vertical ones, due to the normal pressure acting on the joint.

In the variants of the original project and even in other variants with weight and stiffness transformations, the structural joints do not fail. This corresponds to what was initially stated in relation to the adequate behavior of the prefabricated system due to the type of joint between the elements, among other aspects. However, the possibility of cracking of the elements and joints in these variants, as potential seismic damage characteristic of these large-panel prefabricated structures, is not ruled out.

In the experimental investigations of Karthikeyan, Santhi, and Chidambaram (2019) as well as Karthikeyan and Santhi (2019) the prefabricated panels tested, in relation to the monolithic ones, proved to have more resistance, however, they developed cracking in the joints. Although in particular Karthikeyan et al. (2019) achieve efficient load transfer in precast panels, Vaghei et al. (2014) also observed cracking along the panels and the connection of precast walls during the application of an incremental lateral load.

Conclusion

The seismic behavior of precast concrete panel structures in relation to structural joints was analyzed. Multi mass models were used in which the joints between the elements were not modeled explicitly and were assumed to be rigid. The design stresses in the joints were obtained by section cuts. Then, the shear resistance of the concrete, the resistance of the welds that join the projecting steels and the possibility of creep of these steels were evaluated. It is concluded that only in 5-level variants with pathological damage, there is a danger of failure of horizontal and vertical joints.

References

American Concrete Institute [ACI]. (2019). *Building code requirements for structural concrete (ACI 318-19)*. American Concrete Institute. Retrieved from https://www.usb.ac.ir/FileStaff/5526_2020-1-25-11-12-7.pdf

Baykov, V. N., & Sigalov, E. E. (1980). *Estructuras de hormigón armado*. Moscú, RU: MIR.

Benjumea, J., Saiid, M., & Itani, A. (2020). Seismic performance analysis and assessment of a precast bridge computational model. *DYNA, 87*(212), 80-89. DOI: https://doi.org/http://doi.org/10.15446/dyna.v87n212.80143

Chopra, A. K. (2014). *Dinámica de estructuras* (4 ed.). Naucalpan de Juárez, MX: Pearson.

Consejo Superior de Fondonorma [COVENIN]. (2001). *Edificaciones sismorresistentes. Parte 1: Requesitos y Comentarios*. Caracas, VE: COVENIN.

Coronelli, D., & Gambarova, P. G. (2004). Structural assessment of corroded reinforced concrete beams: modeling guidelines. *Journal of Structural Engineering, 130*(8), 1214-1224. DOI: https://doi.org/10.1061/(ASCE)0733-9445(2004)130:8(1214)

Du, Y., Clark, L., & Chan, A. H. C. (2005). Effect of corrosion on ductility of reinforcing bars. *Magazine of Concrete Research, 57*(7), 407-419. DOI: https://doi.org/10.1680/macr.2005.57.7.407

Federal Emergency Management Agency [FEMA]. (1997). *NEHRP guidelines for the seismic rehabilitation of buildings (FEMA 273)*. Washington, DC. Retrieved from https://courses.washington.edu/cee518/fema273.pdf

Federal Emergency Management Agency [FEMA]. (1998). *Handbook for the Seismic Evaluation of Buildings (FEMA 310)*. Washington, DC: Fema.

Federal Emergency Management Agency [Fema]. (2015). *Rapid visual screening of buildings for potential seismic hazards: a handbook* (3rd ed.). Washington, DC: Fema.

Gunawardena, T., Ngo, T. D., Mendis, P., & Kumar, S. (2017). Performance of multi-storey prefabricated modular buildings with infill concrete walls subjected to earthquake loads. *Concrete in Australia, 43*(3), 51-58.

Holly, I., & Abrahoim, I. (2020). Connections and joints in precast concrete structures. *Slovak Journal of Civil Engineering, 28*(1), 49-56. DOI: https://doi.org/10.2478/sjce-2020-0007

Karthikeyan, K., & Santhi, M. H. (2019). Experimental investigation on precast wall connections. *Journal of Advanced Research in Dynamical and Control Systems, 11*(6), 1672-1678.

Karthikeyan, K., Santhi, M. H., & Chidambaram, C. R. (2019). Behaviour of horizontal connections in precast walls under lateral loading. *International Journal of Recent Technology and Engineering, 8*(3), 436-440. DOI: https://doi.org/10.35940/ijrte.C4217.098319

Lewicki, B. (1968). *Edificios de viviendas prefabricadas con elementos de grandes dimensiones*. Arkady, PL: Instituto Eduardo Torroja de la Construccion y del cemento.

McCormac, J. C. (2008). *Diseño de estructuras de acero (Método LRFD)*. CU: Alfaomega.

Oficina Nacional de Normalización [NC]. (2003a). *Densidad de materiales naturales, artificiales y de elementos de construcción como carga de diseño (NC 283: 2003)*. La Habana, CU: NC.

Oficina Nacional de Normalización [NC]. (2003b). *Edificaciones. Cargas de uso. (NC 284: 2003)*. La Habana, CU: NC.

Oficina Nacional de Normalización [NC]. (2017). *Construcciones sismo resistentes. Requisitos básicos para el diseño y construcción (NC 46)*. La Habana, CU: NC.

Polyakov, S. V. (1974). *Design of earthquake resistant structures: basic theory of seismic stability*. Moscow, RU: MIR Publishers.

Socarrás Cordoví, Y. C., & Álvarez Deulofeu, E. R. (2021). Vulnerabilidad sísmica del sistema estructural prefabricado gran panel Soviético en edificios deteriorados y transformados. *Obras y Proyectos, 30*, 60-73. DOI: http://dx.doi.org/10.4067/S0718-28132021000200060

Socarrás Cordoví, Y. C., Álvarez Deulofeu, E. R., & Galbán Rodriguez, L. (2021c). Comportamiento sísmico de edificios del sistema prefabricado Gran Panel Soviético en relación a factores del suelo. *Minería y Geología, 37*(2), 214-230.

Socarrás Cordoví, Y. C., Álvarez Deulofeu, E. R., & Lora Alonso, F. (2021a). Forecasts on the seismic behavior of buildings constructed with the Great Soviet Panel. *DYNA, 88*(216), 145-151. DOI: https://doi.org/10.15446/dyna.v88n216.87946

Socarrás Cordoví, Y. C., Álvarez Deulofeu, E. R., & Lora Alonso, F. (2021b). Changes in the fundamental periods of buildings constructed with the Great Soviet Panel. *ESTOA, 10*(19), 220-235. DOI: https://doi.org/10.18537/est.v010.n019.a12

Socarrás Cordoví, Y. C., Álvarez Deulofeu, E. R., & Moreno Roche, E. (2020c). Repercusiones de las contravenciones estructurales e incremento de peso en el sistema gran panel soviético en Santiago de Cuba. *Revista de Obras Públicas, 3623*, 74-82.

Socarrás Cordoví, Y. C., Álvarez Deulofeu, E. R., & Pupo Sintras, N. (2021d). Structural behavior of construction typologies of the Great Soviet Panel in an area of high seismic danger. *DYNA, 88*(219), 155-161. DOI: https://doi.org/10.15446/dyna.v88n219.97345

Socarrás Cordoví, Y. C., González Díaz, L., & Álvarez Deulofeu, E. R. (2022). Significant reductions in the area in corroded steel and its repercussion in prefabricated large-panel buildings. *Revista Facultad de Ingeniería, 31*(59), e13110. DOI: https://doi.org/10.19053/01211129.v31.n59.2022.13110

Socarrás Cordoví, Y. C., González Díaz, L., Álvarez Deulofeu, E. R., González Fernández, M., & Roca Fernández, E. (2020b). Evaluación de la calidad del hormigón en edificaciones construidas con el sistema prefabricado gran panel soviético. *Tecnología Química, 40*(2), 288-302.

Socarrás Cordoví, Y. C., González Díaz, L., Álvarez Deulofeu, E. R., González Fernández, M., Roca Fernández, E., & Torres Shoembert, R. (2020a). Valuation of the durability of the concrete used in the precast great soviet panel system. *Revista Facultad de Ingeniería, 29*(54), e10486. DOI: https://doi.org/10.19053/01211129.v29.n54.2020.10486

Socarrás Cordoví, Y., & Vidaud Quintana, I. N. (2017). Desde la tecnología del prefabricado actual hasta la prefabricación contra pedido. *Ciencia en su PC, 1*, 104-115.

Vaghei, R., Hejazi, F., Taheri, H., Jaafar, M. S., & Ali, A. A. A. (2014). Evaluate performance of precast concrete wall to wall connection. *APCBEE Procedia, 9*, 285-290. DOI: https://doi.org/10.1016/j.apcbee.2014.01.051

Zhou, J., Zhi, X., Fan, F., Jiao, A., & Qian, H. (2020). Experimental and numerical investigation on failure behavior of ring joints in precast concrete shear walls. *Advances in Structural Engineering, 23*(1), 118-131. DOI: https://doi.org/10.1177/1369433219864