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Analysis of the Structural Elements of a Condominium with a Swimming Pool in the City of Naberezhnye Chelny under Various External Influences

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Abstract

The article discusses environmental issues as well as two design options of a condominium: a design scheme with a 6000x6000 m grid of columns and a design scheme with a 6000x12,000 m grid of columns. The calculation results for Naberezhnye Chelny are given, excluding dynamic loads with an elastic foundation and considering wind pulsations on a rigidly restrained foundation. The analysis of the calculation of the load-bearing systems of a multi-storey building with a monolithic frame is presented, aimed at the maximum possible reduction in the cross-section of the frame elements transmitting the load, as well as their number, respectively, and at reducing costs. Considering all the requirements and norms, the structural scheme of the monolithic frame of the building, designed on an elastic foundation, which was optimal in all parameters, was developed, subject to the given soil options. Wind and snow loads were calculated. Loads of the dead weight of structures, roof and floor were also considered. The calculation for the impact of dynamic loads, namely the pulsations of the wind, given along the axes of coordinates X and Y [2; 3]. Loads were determined with an elastic foundation and a grid of 6x6 m and 6x12 m columns excluding wind pulsations and loadings, considering the dynamic load of wind pulsations with rigid support of the foundation. The strength test was performed for three combinations. Calculations were made for maximum and minimum efforts and stresses. The selection of reinforcement with an elastic foundation and with dynamic loads of wind pulsations has been performed. Calculation and selection of supporting structures and required reinforcement were performed using SCAD software. The calculation results were also verified here. The structure of a multi-storey condominium with a 6 by 6 column grid is designed in such a way that it can withstand the acting forces and loads, i.e. mobilize reaction forces that guarantee the balance of the frame with an elastic foundation.

As a result of the above calculations, a positive assessment was given of the possibility of designing a building with a calculation scheme with a 6000x6000 m grid of columns.

Keywords

Condominium; Building Structures; Dynamic Loads; Monolithic Frame; The Dead Weight Of The Structure; Wind Loads

Introduction

Residential and public buildings and structures are durable objects and are very often exposed to various external influences during operation, including climatic loads (wind, snow). All this causes additional efforts from new operational loads and affects the process of structural elements of the building.

The most important task of the construction industry is to reduce the cost of structures of buildings and structures while observing the main criteria for the bearing capacity of structures, as well as safety indicators of buildings and structures under various external influences. Optimal design is one of the ways to solve this problem. The authors carried out the calculation for the stability of a monolithic multi-storey building, based on the analysis in SCAD Office 11.5. Measures are proposed to improve the strength of the structure. The relevance is determined by the correct choice of the parameters of the sections of the monolithic construction of a condominium for 150 apartments and the construction of the pool building covering according to the given internal efforts.

Overall, in this study, the fundamental features of a condominium with a swimming pool in the city of Naberezhnye Chelny was analyzed, and finally, the basis for the choice of the design scheme was a 150-apartment projected condominium building. Both scientific-practical and architectural-aesthetic interest is presented.

Methods

The object of research for the subsequent design was the structural elements, namely the reinforced concrete monolithic structures of the condominium. To select the optimal design scheme of the condominium frame in all parameters, considering the requirements and norms, it is necessary to calculate the wind and snow loads in accordance with the regulatory documents.
The source data for the calculations was the projected condominium building: the degree of responsibility of the building – I; the degree of fire resistance – II; functional fire hazard class – F 1.3; constructive fire hazard class – C. The building plan dimensions are 72x72 m with a different height. The height of the premises is 4.2 m and 6.2 m.
Design schemes have been designed with a grid of columns 6000*6000 mm and 6000*12,000 mm, in a monolithic frame, secondary beams 250x300x6000 mm, main beams 250x500x6000 mm, a monolithic floor, a foundation slab, columns 400x600x4200 mm and 400x400x400, 300x300x4200 mm are used.

Results and Discussion

In order to prevent the roof collapse during the operation of the building, the design and calculation stage should consider:
dead load (SCAD-calculated);
the weight of the floor and the roofing pie;
temporary load in 3 loading options;
leeward and windward wind load;
snow load in two loading options.
The calculated load of the roofing pie weight is $1.1\ kN|m^2$ and floor weight is $1.2\ kN/m^2$.

Fig. 1: Designed Condominium Building.

Temporary load on the ceiling in 3 loading options:
1. Full floor loading is 4.8 kN/m^2;
2. Staggered, close to architecture and types of premises is 5.4 kN/m^2;
3. The perpendicular load along the spans is 5.4 kN/m^2, which allows the load to be distributed so as to reduce the cost of reinforcement.

The leeward and windward wind load is calculated. The calculation was performed using the WEST program. The calculation was carried out according to the design standards SP 20.13330.2016 “Loads and Impacts”.
- from the windward side:

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the frame spacing is 6 m; the windward load on the columns will be:
$w_I{}^н$ =0.247 kN/m^2*6 m=1.482 kN/m,
$w_2{}^н$ =0.584 t/m^2*6 m=3.54 kN/m.
- from the leeward side:
the frame spacing is 6 m; the leeward load on the columns will be:
$w_I{}^н$ = -0.185 kN/m^2*6 m= -1.11 kN/m,
$w_2{}^н$ = -0.438 kN/m^2*6 m= -2.64 kN/m,
Snow load in two loading options:
a) uniform snow load equal to 1.21 kN/m^2 over the entire surface;
b) in the coated areas adjacent to the ventilation shafts and other superstructures that rise above the roof, an increased load is indicated, according to SP 20.13330.2016 p.Г.11, equal to 1.21*2=2.42 kN/m^2.

Results of loads with an elastic foundation and a 6x6 m column grid excluding wind pulsations

The maximum effort values are presented based on the calculation results in SCAD Soft. The results are shown in Table 1.

Table 1: Maximum Force Values

MAXIMUM FORCES / STRESSES / IN THE ELEMENTS OF THE CALCULATION SCHEME								
Name	max +				max -			
	Value	Elem.	Sect.	Heat.	Value	Elem.	Sect.	Heat.
N	58.1162	202867	1	1	58.1162	202867	1	1
Mk	2.43052 1	204271	1	1	2.6008	1470	1	1
My	78.3498	204175	1	1	82.042	1441	1	1
Qz	228.282	1441	1	1	183.9820	4175	3	1
Mz	88.0524	1441	1	1	-50.775	1458	1	1
Qy	246.944	1441	1	1	-173.96	1458	1	1
NX	746.802	8539	1	1	-3096.2	3838	1	1
NY	669.675	19874	1	1	-3140.3	200073	1	1

The maximum displacement is δmax = 180 mm
$\delta\max = 180\text{mm} < \left[\frac{f}{l}\right] = \frac{l}{300} = \frac{72000}{300} = 240$ mm, **the stiffness condition is fulfilled.**

The results of loads considering the dynamic load of wind pulsations with the rigid fastening of the base and a 6x6m grid of columns

The base works according to the cantilever scheme. All temporary ceiling loads are mutually exclusive; combinations allow for mutual exclusion of temporary full ceiling loads, snow load and wind pulsations. The statically set wind is only a part of the wind loads; it is neglected in this calculation, the wind pulsations specified using SP 20.13330 "Calculated combinations of forces and loads" are considered.

Creating a combination of loads, the values of the design loads were multiplied by the combination coefficients presented in Table 4, according to SP 14.13330.2016.

The results of cumulative displacement for various combinations of force and load combinations are given below.

The maximum displacement is δmax = 95.85 mm
$\delta\max = 95{,}85\text{ mm} < \left[\frac{f}{l}\right] = \frac{l}{300} = \frac{72000}{300} = 240$ mm, the stiffness condition is fulfilled, pulsing deformation is +x.

The maximum displacement is δmax = 60.88 mm
$\delta\max = 60.88\text{ mm} < \left[\frac{f}{l}\right] = \frac{l}{300} = \frac{72000}{300} = 240$ mm, the stiffness condition is fulfilled, pulsing deformation is – x.

The maximum displacement is δmax = 99.18 mm
$\delta\max = 99.18\text{ mm} < \left[\frac{f}{l}\right] = \frac{l}{300} = \frac{72000}{300} = 240$ mm, the stiffness condition is fulfilled, pulsing deformation is + y.

The maximum displacement is δmax = 52.04 mm
$\delta\max = 52.04\text{ mm} < \left[\frac{f}{l}\right] = \frac{l}{300} = \frac{72000}{300} = 240$ mm, the stiffness condition is fulfilled, pulsing deformation is y.

Further, a check was made for the maximum permissible horizontal displacement.

Strength test for 3 combinations.

1. The X-axis displacement of the building (combination No. 1) under the combined effect of vertical and horizontal loads is $\delta\max x - axis = 54.83\text{ mm} < \left[\frac{f}{h}\right] = \frac{h}{500} = \frac{67200}{500} = 134.4\ mm,\ \delta\max y - axis =$

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$76.23\text{ mm} < \left[\frac{f}{h}\right] = \frac{h}{500} = \frac{67200}{500} = 134.4\ mm$, **the stiffness condition is fulfilled**

2. The X-axis displacement of the building (combination No. 2) under the combined effect of vertical and horizontal loads is $\delta \max x - axis = 80.49\text{ mm} < \left[\frac{f}{h}\right] = \frac{h}{500} = \frac{67200}{500} = 134.4\ mm$, $\delta \max y - axis = 82.79\text{ mm} < \left[\frac{f}{h}\right] = \frac{h}{500} = \frac{67200}{500} = 134.4\ mm$, **the stiffness condition is fulfilled**

3. The Y-axis displacement of the building (combination No. 3) under the combined effect of vertical and horizontal loads is $\delta \max x - axis = 24.33\text{ mm} < \left[\frac{f}{h}\right] = \frac{h}{500} = \frac{67200}{500} = 134.4\ mm$, $\delta \max y - axis = 25.06\text{ mm} < \left[\frac{f}{h}\right] = \frac{h}{500} = \frac{67200}{500} = 134.4\ mm$, **the stiffness condition is fulfilled**

Results of SCAD calculation - minimax of displacement

Minimax of displacement								
Factor	Maximum values				Minimum values			
	Value	Unit	Load	Form	Value	Unit	Load	Form
X	98.506	1772	14	LS+SD	-95.19	1772	13	LS+SD
Y	51.822	1822	16	LS+SD	-60.627	1822	15	LS+SD
Z	7.64	1821	14	LS+SD	-7.376	1821	13	LS+SD
Ux	0.921	217	15	LS+SD	-0.772	217	16	LS+SD
Uy	1.715	272	14	LS+SD	-1.658	272	13	LS+SD
Uz	0.496	1672	14	LS+SD	-0.48	1672	13	LS+SD

Results of SCAD calculation - minimax of forces and stresses

Minimax of forces and stresses										
Factor	Maximum values					Minimum values				
	Value	Element	Section	Load	Form	Value	Element	Section	Load	Form
N	161.571	1459	1	13	LS+SD	-167.489	1459	1	14	LS+SD
Mk	2.035	1442	1	14	LS+SD	-1.964	1442	1	13	LS+SD
My	45.116	1442	1	13	LS+SD	-46.73	1442	1	14	LS+SD
Qz	117.644	1441	1	14	LS+SD	-113.677	1441	1	13	LS+SD
Mz	25.365	1441	1	13	LS+SD	-26.185	1441	1	14	LS+SD
Qy	74.914	1441	1	13	LS+SD	-77.35	1441	1	14	LS+SD

The results of SCAD calculations of the construction of a residential condominium with and without wind pulsations are presented in Table 2.

Table 2: Calculation Results of the Housing Complex Structural Element

Travels:	Excluding wind pulsation with elastic base	Considering the pulsation of the wind with the base, operating according to the cantilever scheme
Z-axis (maximum flexure)	-180.381 mm	7.64 mm
Y-axis	41.27 mm	-60.627 mm
X-axis	17.59 mm	-98.506 mm
Reinforcement, cross-sectional dimensions of reinforced concrete elements:		
Floor slab	δ = 120 mm, Main reinforcement Ø3-5 mm	δ = 120 mm, Main reinforcement Ø3-5 mm
Columns	400x600, 400x400 reinforcement Ø18, Ø25, Ø28, Ø32, spacing 100 mm, longitudinal A400 and transverse A240 reinforcement class.	The cross-sectional dimensions of most of the columns were not sufficient.
Foundation slab	δ = 900 mm, Reinforcement Ø5, spacing 100, Bp500	The foundation slab is tested for strength; reinforcement is required only in some areas of the foundation slab. The rigidity of the foundation slab operating according to the cantilever

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		scheme is provided. Reinforcement BP500.
Main beams	h = 500 mm, b = 100 mm Reinforcement Ø15-32, longitudinal A400 and transverse A240 reinforcement class.	The main operating reinforcement - d18-28. Longitudinal A400 and transverse A240 reinforcement class. However, most of the building's main beams failed the strength test. The cross-section of the beams must be increased.
Secondary beams	h = 300 mm, b = 100 mm Reinforcement Ø14-32, longitudinal A400 and transverse A240 reinforcement class.	The main operating reinforcement - d22,25,28 and 32. Longitudinal A400 and transverse A240 reinforcement class. A few of the building's main beams failed strength test. The cross-section of the secondary beams must be increased.

As the initial data for calculating the structural scheme with a design scheme of 6000x12000 mm, the same structural elements were taken as for the 6000x6000 mm grid of columns.
The calculated load of the roofing pie weight is 1.1 kN|$m^{2,}$ and floor weight is 1.2 kN/m^2.
Temporary load on the ceiling in 3 loading options [5]:
- full floor loading is 4.8 kN/m^2;
- staggered, close to architecture and types of premises is 5.4 kN/m^2;
- the perpendicular load along the spans is 5.4 kN/m^2, which allows the load to be distributed so as to reduce the cost of reinforcement.

Leeward and windward wind load was calculated. The calculation was performed using the WEST program. The calculation was carried out according to the design standards SP 20.13330.2016.
- from the windward side

The frame spacing is 6 and 12 m; the windward load on the columns will be:
X-axis
$w_I^{н}$ =0.247 kN/m^2*6 m=1.482 kN/m, $w_2^{н}$ =0.584 t/m^2*6 m=3.54 kN/m
Y-axis
$w_I^{н}$ =0.247 kN/m^2*6 m=2.964 kN/m, $w_2^{н}$ =0.584 t/m^2*6 m=7.008 kN/m
- from the leeward side

The frame spacing is 6 and 12 m; the leeward load on the columns will be:
X-axis
$w_I^{н}$ = -0.185 kN/m^2*6 m= -1.11 kN/m, $w_2^{н}$ = -0.438 kN/m^2*6 m= -2.64 kN/m
Y-axis
$w_I^{н}$ = -0.185 kN/m^2*6 m= -2.22 kN/m, $w_2^{н}$ = -0.438 kN/m^2*12 m= -5.256 kN/m
Snow load in two loading options. The calculation was carried out according to the design standards SP 20.13330.2016.
a) uniform snow load equal to 1.21 kN/m^2 over the entire surface;
b) in the coated areas adjacent to the ventilation shafts and other superstructures that rise above the roof, an increased load is indicated, according to SP 20.13330.2016 p.Г.11, equal to 1.21*2.5=3.035 kN/m^2.
Results of loads with an elastic foundation and a 6x12 m column grid excluding wind pulsations. Minimax of forces and stresses (combinations) are shown in Table 3.

Table 3: Maximum Forces and Stresses

Minimax of forces and stresses (combinations)								
Factor	Maximum values				Minimum values			
	Value	Element	Section	Combination	Value	Element	Section	Combination
N	139.316	222297	1	1	-3182.347	794	1	1
Mk	5.386	806	1	1	-6.036	269674	1	1
My	233.645	795	1	1	-243.198	801	1	1
Qz	631.512	801	1	1	-602.056	795	1	1
Mz	353.555	801	1	1	-103.657	805	3	1

Minimax of forces and stresses (combinations)								
Factor	Maximum values				Minimum values			
	Value	Element	Section	Combination	Value	Element	Section	Combination
Qy	1014.167	801	1	1	-334.219	794	1	1
NX	1183.706	198089	1	1	-7598.402	1101	1	1
NY	1518.176	1783	1	1	-4802.988	1565	1	1
TXY	4052.06	1678	1	1	-4825.535	1565	1	1
MX	733.361	205625	1	1	-192.931	16024	1	1
MY	852.676	205625	1	1	-325.298	206900	1	1
MXY	120.51	199755	1	1	-233.004	201659	1	1
QX	1385.535	209189	1	1	-1036.826	205702	1	1
QY	1177.326	205625	1	1	-1396.327	209189	1	1
RZ	0	0	0	0	-87.797	210883	1	1

The maximum vertical displacement is δmax = 354.0381 mm $< \left[\frac{f}{l}\right] = \frac{l}{300} = \frac{67200}{300} = 224\ mm$, **the stiffness condition is not met, and the building does not meet the strength requirements.**
The displacement of the building under the combined effect of vertical and horizontal loads is $\delta\max x - axis = 56.84$ mm $< \left[\frac{f}{l}\right] = \frac{h}{500} = \frac{67200}{500} = 134.4\ mm$, $\delta\max y - axis = 86.83$ mm $< \left[\frac{f}{l}\right] = \frac{h}{500} = \frac{67200}{500} = 134.4\ mm$.

The results of loads considering the dynamic load of wind pulsations with rigid fastening of the base
The base works according to the cantilever scheme. All temporary ceiling loads are mutually exclusive; combinations allow for mutual exclusion of temporary full ceiling loads, snow load and wind pulsations. The statically set wind is only a part of the wind loads; it is neglected in this calculation, the wind pulsations specified using SP 20.13330.2016 "Calculated combinations of forces and loads" are considered.
Creating a combination of loads, the values of the design loads were multiplied by the combination coefficients presented in Table 4 according to SP 14.13330.2016.
The results of cumulative displacement for various combinations of force and load combinations are given below.
The maximum displacement is δmax = 9,24 mm
δmax = 9.24 mm $< \left[\frac{f}{l}\right] = \frac{l}{300} = \frac{67200}{300} = 224\ mm$, **the stiffness condition is fulfilled**, pulsing deformation is +x.
The maximum displacement is δmax = 6.67 mm
δmax = 6.67 mm $< \left[\frac{f}{l}\right] = \frac{l}{300} = \frac{67200}{300} = 224\ mm$, **the stiffness condition is fulfilled**, pulsing deformation is – x.
The maximum displacement is δmax = 23.44 mm
δmax = 23.44 mm $< \left[\frac{f}{l}\right] = \frac{l}{300} = \frac{67200}{300} = 224\ mm$, **the stiffness condition is fulfilled**, pulsing deformation is +y.
The maximum displacement is δmax = 19.28 mm
δmax = 19.28 mm $< \left[\frac{f}{l}\right] = \frac{l}{300} = \frac{67200}{300} = 224\ mm$, **the stiffness condition is fulfilled**, pulsing deformation is – y.
Further, for a more accurate result, it is necessary to check for the maximum permissible horizontal displacements for three combinations of loads.
Strength test for 3 combinations.

1. The displacement of the building (combination No. 1) under the combined effect of vertical and horizontal loads is $\delta\max \mathrm{x} - \mathrm{axis} = 131.38$ mm $> \left[\frac{f}{l}\right] = \frac{h}{500} = \frac{67200}{500} = 134.4\ mm$, $\delta\max y - axis = 179.41$ mm $> \left[\frac{f}{l}\right] = \frac{h}{500} = \frac{67200}{500} = 134.4\ mm$, *the conditions are not met.*
2. The displacement of the building (combination No. 2) under the combined effect of vertical and horizontal loads is $\delta\max \mathrm{x} - \mathrm{axis} = 130.0$ mm $> \left[\frac{f}{l}\right] = \frac{h}{500} = \frac{67200}{500} = 134.4\ mm$, $\delta\max y - axis = 174.22$ mm $> \left[\frac{f}{l}\right] = \frac{h}{500} = \frac{67200}{500} = 134.4\ mm$, *the conditions are not met.*
3. The Y-axis displacement of the building (combination No. 3) under the combined effect of vertical and horizontal loads is $\delta\max \mathrm{x} - \mathrm{axis} = 128.53$ mm $> \left[\frac{f}{l}\right] = \frac{h}{500} = \frac{67200}{500} = 134.4\ mm$, $\delta\max y - axis = 201.36$ mm $> \left[\frac{f}{l}\right] = \frac{h}{500} = \frac{67200}{500} = 134.4\ mm$, the conditions are not met

The results of SCAD calculations of the building with and without wind pulsations are presented in Table 4.

Table 4: Calculation Results

Displacements:	Excluding wind pulsation with an elastic base	Considering the pulsation of the wind with the base, operating according to the cantilever scheme, for 3 load combination options
Z-axis (maximum flexure)	-180.381 mm	-146.17 mm
Y-axis	41.27 mm	-201.359 mm
X-axis	17.59 mm	-131.38 mm
Reinforcement, cross-sectional dimensions of reinforced concrete elements:		
Floor slab	δ = 600 mm, Main reinforcement Ø3-5 mm, Bp500	δ = 600 mm, Main reinforcement Ø3-5 mm, Bp500
Columns	The cross-sectional dimensions of most of the columns were not sufficient.	The cross-sectional dimensions of most of the columns were not enough.
Foundation slab	δ = 900 mm, Reinforcement Ø5, spacing 100, Bp500	δ = 900 mm. The foundation slab is tested for strength. The rigidity of the foundation slab operating according to the cantilever scheme is provided. Reinforcement Bp500.
Main beams	h = 500 mm, b = 250 mm Reinforcement Ø25, 28 and 32, longitudinal A400 and transverse A240 reinforcement class.	The main operating reinforcement - d14-22. Longitudinal A400 and transverse A240 reinforcement class.
Secondary beams	h = 300 mm, b = 250 mm Reinforcement Ø10-18, longitudinal A400 and transverse A240 reinforcement class.	Main operating reinforcement Ø10-18, longitudinal A400 and transverse A240 reinforcement class.

Summary

The results of the study show that the optimal constructive schemes of the condominium in all parameters have been developed. The stability of the structure with a 6000x6000 mm grid of columns on an elastic foundation is ensured. The structure of a multi-storey condominium with a grid of 6x6 m columns is designed so that it can withstand the acting forces and loads, i.e. mobilize reaction forces that guarantee the balance of the frame with an elastic foundation. Based on this, it can be concluded that with a 6x6 m column grid, the stability of the structure in the version with an elastic foundation is fully ensured. According to all checks for vertical and horizontal movements, the task of ensuring that the building operates under the influence of wind loads that would meet the requirements of reliability and suitability for normal operation throughout its entire service life has been solved.

Horizontal and vertical displacement was checked with a rigidly fastened foundation, with a grid of 6x6 m and 6x12 m columns.

The frame meets all the requirements for stability, but not for the rigidity of vertical structures, which in turn require significant cross-sectional areas of the supports, which limit the usable floor area and increase the construction costs of this facility.

A monolithic structure with a 6x12 m grid of columns failed maximum deflection test; therefore, it is necessary to strengthen the frame either by installing stiffeners or by strengthening the columns and reducing the span, which will lead to additional costs. Therefore, this option is not applicable to construction.

The calculations revealed that the selected structural scheme of the monolithic frame of the condominium with a 6000x6000 mm grid of columns is aimed at the maximum possible reduction in the cross-section of the frame elements transmitting the load, as well as their number, respectively, and at reducing costs.

Conclusions

The work involved a comparative analysis of the calculations of the structural scheme of a condominium monolithic frame with 6000x6000 mm and 6000x12000 mm grid of columns. The stability of the structure with a 6000x6000 mm grid of columns on an elastic foundation is ensured.

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The projected monolithic multi-storey building on a rigidly supported foundation slab, considering dynamic loads in the form of wind pulsations from the windward and leeward sides, does not meet the strength requirements, since, according to regulatory requirements, the cross-section is small for the required maximum percentage of reinforcement. Only by increasing the cross-sections, increasing the strength, and the weight of the supporting structures, it is possible to increase the stability of the building, considering the pulsation of the wind. The structure may be durable, but this solution will be economically disadvantageous because both the mass and the dynamic load can increase even more.

In conclusion, we should note that with a 6x6 m column grid, the stability of the structure in the option with an elastic foundation is fully ensured, according to all checks for vertical and horizontal displacements.

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