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Structural Design of the Tallest Concrete Tower in Korea, the # Star City

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Abstract

Residential high-rise buildings, such as apartments and condominiums, were introduced to provide multiple residential spaces in urban and suburban areas for their economic advantage and convenience for commuting to urban business districts. Old-fashioned apartments were basically stacked rectangular box buildings. The typical floor framing system was a concrete flat slab with beams, supported on columns and bearing walls conveniently hidden in the partitions between rooms and apartment units. It was very practical and economical since all every apartment units were vertically stacked and had the same layout, thus allowing simple vertical risers for MEP services.

Recently, prospective tenants have started demanding the ability to customize their interior designs to accommodate varying family sizes and lifestyles. More functional and practical layouts were preferred to old standardized apartment modules. This space planning conflicted with the traditional column and wall layouts, spurring demand for a new flexible structural system in order to satisfy tailored requests of tenants. A concrete flat plate floor surrounding a central core wall system became appealing because this system provides flexibility to tenants to modify the interior space to suit their own lifestyles. The tallest concrete tower in Korea, The # Star City project, has adopted this long-span structural system as described above.

Keywords: Residential high-rise building; ability to customize their interior designs; new flexible structural system; concrete flat plate floor surrounding central core wall system; the tallest concrete tower in Korea

1. Introduction

The # (sharp) Star City is a residential complex located on the banks of the lower Han River, across from the Kun-Kook University in Seoul, Korea. It is currently under construction with POSCO E&C and will be the tallest concrete tower in Korea.

The project consists of three residential towers (58, 50, and 45 stories) and one mixed-use 35-story tower, as well as four below grade basement levels intended to be used for parking, retail, health, mechanical, and electrical facilities. The total gross area of the four towers is approximately 238,000 m^2 (2.56M ft^2).

The typical floor framing system consists of a cast-in place flat plate with a slab thickness of 250 mm (10 in.) supported on reinforced concrete columns and core walls. Flat plate floors span between the columns and central core walls without the use of intermediate beams, spandrel beams, drop panels or capitals.

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Fig. 1. View of The # Star City

Not only does this type of system reduce the floor to floor height requirements, but it also saves considerable construction cost and allows a short construction cycle. Saving construction time was possible due to the simple flat formwork and shoring as well as repetition of the formwork pattern at every floor.

The primary lateral load resisting system consists of reinforced concrete shear walls. Flat plate participation as a moment frame has also been considered to resist the lateral loads in addition to the core shear wall. This project was designed with the consideration of critical issues for high-rise residential towers including limiting building acceleration for occupant comfort and addressing differential shortening between columns and core walls.

2. Structural Material Properties

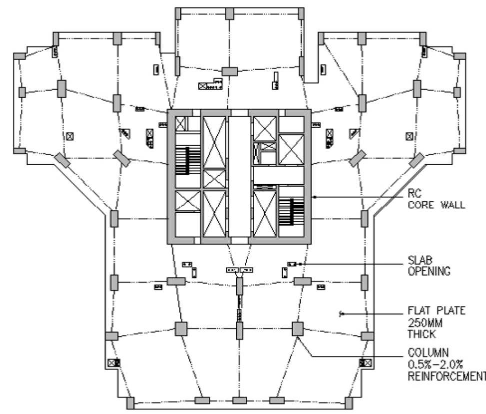
The primary structural material used in the four towers was concrete with strengths varying from f'c = 300 kgf/cm^2 (4.26 ksi) to f'c = 600 kgf/cm^2 (8.52 ksi) depending on the height of the buildings. For the tallest tower, Tower A, a high strength concrete of f'c = 600 kgf/cm^2 (8.52 ksi) was utilized for the corewall and columns below the 10th floor. For the other three towers, strength was limited to 500 kgf/cm^2 (7.11 ksi).

Per ACI guidelines, the maximum allowable ratio of concrete strength for columns to that of the slab was capped at 1.4. Thus, the concrete strengths of floor slab vary from f'c = 430 kgf/cm^2 (6.1 ksi) where corewall or column concrete strength used was f'c = 600 kgf/cm^2 (8.52 ksi) to f'c = 300 kgf/cm^2 (4.26 ksi). The strength of the reinforcing steel was fy = 4,000 kgf/cm^2 (57 ksi).

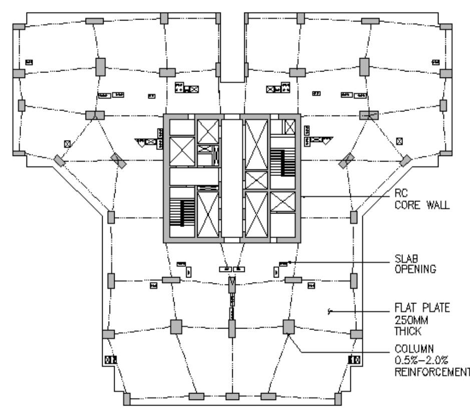
In response to the relatively large shear and flexural forces induced in the link beams across an opening in the core walls, structural steel wide flange members encased in concrete beams were used. Where shear and flexural forces were not excessive, reinforced concrete beams with mild reinforcement were used as link beams. The strength of steel plates and rolled sections was fy = 3,300 kgf/cm^2 (46.9 ksi).

One of the caveats of using high strength concrete is performance in fire. High strength concrete is more vulnerable to spalling. Tests on the performance of high strength concrete show that fire-resistance is an issue due to the low water-cement ratio for high strength concrete (concrete strength equal to or greater than f'c = 700 kgf/cm^2 (10 ksi)). However, since the maximum strength used for this project was less than the threshold limit, the required fire-resistance was achieved.

One of the architectural requirements was to maintain the same apartment layout at every floor for each corresponding unit, including the many different column sizes and shapes on one floor, carrying the column sizes throughout the height of the building. Therefore, in lieu of reducing the column size for lighter loads towards the top of the building, both the steel reinforcement quantity and concrete strength have been reduced for columns at the upper floors. During the value engineering process, the column steel reinforcing ratio was reduced to as low as 0.5% of the gross area. Per ACI, it was checked for capacity based on a reduced area of concrete corresponding to a 1% steel area with the balance of the concrete considered as architectural and not contributing to the strength of the column.



(a) Tower A and Tower B



(b) Tower C and Tower D

Fig. 2. Typical Floor Framing Plans of Four Towers

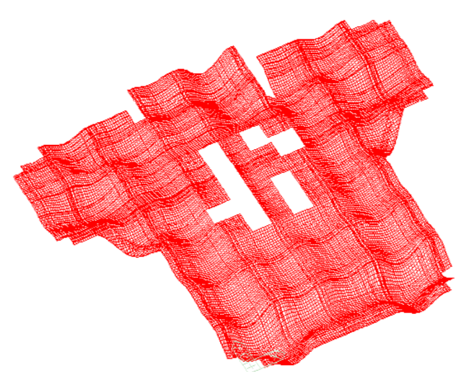


Fig. 3. Floor Slab Long-term Deflection

3. Gravity Load Design

The traditional Korean home culture is sitting on the floor and sleeping on futons. For comfort the floors should be heated during the winter. Nowadays some people prefer to sit on couches and sleep on beds, but they still prefer to walk on radiant heated floors. Also, in Korea residential towers are seen as luxurious rather middle-class. Residents want to have fancy chandeliers at the center of their living rooms and built-in customized furniture in every room. Including the separate radiant floor fill and special ceiling treatment, the typical floor/ceiling construction is about double the depth of what would be in apartment buildings in the United States. Moreover, floor slabs were designed to accommodate the superimposed floor dead loads, which are much greater than the general practice in the United States.

The typical residential floor consists of 250 mm (10 in.) floor slab, 120 mm (5 in.) radiant floor finish, 210 mm (8 in.) ceiling and lights, which yields a total depth of construction equal to 580 mm (23 in.). Since the typical floor to floor height is 3050 mm, the clear ceiling height is 2470 mm. Table 1 below summarizes the representative uniform floor loads used in the design of The # Star City towers. Superimposed dead load includes floor finishes, ceiling, partition walls, permanent mechanical equipment weight, and electric equipment weight. Live loads are based on the Korean Building Code, 2000.

Table 1. Uniformed Floor Loads (kgf/m^2)

Floor Type	CDL	SDL	LL	Notes
Roof	600	365	200	LL=500 @MEP
Residential	600	370	200 R	SDL = 200 @partition
Mechanical	840	365	1000	LL= actual wt.
Residential above Mech.	720	600	200 R	SDL =250 @hung load
Ground-Interior	720	765	500	Double Slab with EPS Block
Ground-Exterior	720	1905	1600	1m thick topping wt.
Basement Parking	720	90	600	
Basement Retail	720	150	1000	
Basement MEP	720	460	1000	

Note: "R" indicates that the corresponding live load is reducible.

4. Structural Analysis Of Lateral Load Resisting System

First, the lateral load resisting systems were analyzed based on the code stipulated wind loads. Since each of the towers has a fairly high slenderness ratio, their lateral stiffness and the accelerations due to wind loads were the controlling parameters which helped to determine the most appropriate lateral load resisting system.

Later, an independent wind tunnel study was performed to verify the order of magnitude of wind pressure and to determine the dynamic behavior of the towers due to wind. The wind tunnel study results revealed a substantial reduction in the wind pressures compared with those based in accordance with the Korean Building Code, thus mitigating the concerns regarding the building deflections and accelerations due to wind loads. Consequently, the seismic loads exceeded the wind loads for all four towers.

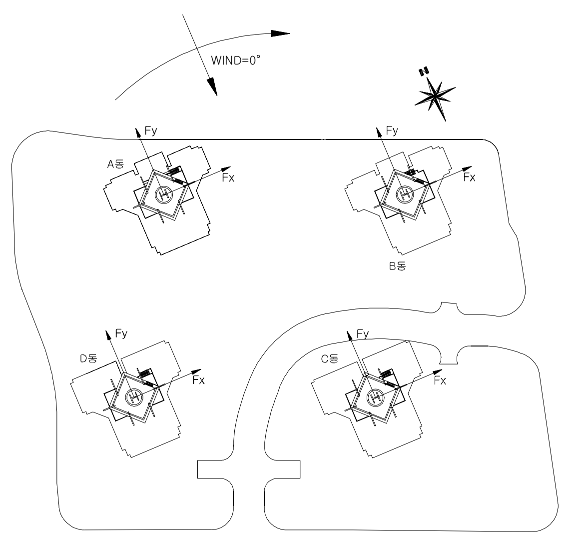


Fig. 4. Wind Tunnel Testing Model (1/400) & Plan View

A summary of results for the performance of the four towers subject to lateral loads (both wind and seismic) with brief tower descriptions can be found in Table 2 through Table 5.

Table 2. Tower A – Lateral Load Resisting System Performance

Rooftop Elevation	193.2 m
Top of Mat Elevation	-18.2 m
No. of Floors Above Grade	58
No. of Floors Below Grade	5
Typical Floor-to Floor Height	3.05 m
Aspect Ratio (Overall Height / Core)	13.5 in X-Dir 14.4 in Y-Dir
Wind Base Shear	1,029 Ton in X-Dir 1,115 Ton in Y-Dir
Seismic Base Shear	2,175 Ton
Overall Drift Due To Wind Loads	H/940 < H/450 in X-Dir H/1024 < H/450 in Y-Dir
Inter-story Drift To Seismic Loads	$H_{FL}/103 < H_{FL}/66$ in X-Dir $H_{FL}/120 < H_{FL}/66$ in Y-Dir

Table 3. Tower B - Lateral Load Resisting System Performance

Rooftop Elevation	125.85 m
Top of Mat Elevation	-19.2 m
No. of Floors Above Grade	35
No. of Floors Below Grade	5
Typical Floor-to Floor Height	3.05 m at Residential Floor 3.40 m at Officetel Floor
Aspect Ratio (Overall Height / Core)	9.5 in X-Dir 10.0 in Y-Dir
Wind Base Shear	362 Ton in X-Dir 594 Ton in Y-Dir
Seismic Base Shear	1,554 Ton
Overall Drift Due To Wind Loads	H/3230 < H/450 in X-Dir H/2440 < H/450 in Y-Dir
Inter-story Drift To Seismic Loads	$H_{FL}/166 < H_{FL}/66$in X-Dir $H_{FL}/172 < H_{FL}/66$ in Y-Dir

Table 4. Tower C - Lateral Load Resisting System Performance

Rooftop Elevation	168.25 m
Top of Mat Elevation	-16.2 m
No. of Floors Above Grade	50
No. of Floors Below Grade	4
Typical Floor-to Floor Height	3.05 m
Aspect Ratio (Overall Height / Core)	12.0 in X-Dir 13.0 in Y-Dir
Wind Base Shear	842 Ton in X-Dir 969 Ton in Y-Dir
Seismic Base Shear	2,126 Ton
Overall Drift Due To Wind Loads	H/1204 < H/450 in X-Dir H/1274 < H/450 in Y-Dir
Inter-story Drift To Seismic Loads	$H_{FL}/102 < H_{FL}/66$ in X-Dir $H_{FL}/129 < H_{FL}/66$ in Y-Dir

Table 5. Tower D- Lateral Load Resisting System Performance

Rooftop Elevation	154.8 m
Top of Mat Elevation	-16.7 m
No. of Floors Above Grade	45
No. of Floors Below Grade	4
Typical Floor-to Floor Height	3.05 m
Aspect Ratio (Overall Height / Core)	11.5 in X-Dir 12.2 in Y-Dir
Wind Base Shear	645 Ton in X-Dir 764 Ton in Y-Dir
Seismic Base Shear	1,941 Ton
Overall Drift Due To Wind Loads	H/1592 < H/450 in X-Dir H/1823 < H/450 in Y-Dir
Inter-story Drift To Seismic Loads	$H_{FL}/120 < H_{FL}/66$ in X-Dir $H_{FL}/149 < H_{FL}/66$ in Y-Dir

As shown in the tables, building deflections and inter-story drifts were far below their limits. Wind tunnel testing results also showed that the building accelerations ranged from 0.6 cm/s^2 to 1.42 cm/s^2 for wind loads with a 5-year recurrence interval. These are imperceptible to most people and thus acceptable in accordance with ISO 6897 and the AISC Design Guide.

The design of the lateral system was based on several parameters including, but not limited to, the stiffness of the structure, center of rigidity and the dynamic properties of the structure. The primary lateral load resisting system of the towers was a central concrete corewall system. Corewall thickness varies from 1000 mm to 500 mm for Tower A, from 600 mm to 400 mm for Tower B, from 800 mm to 500 mm for Tower C, and from 700 mm to 500 mm for Tower D.

In an effort to maximize the tenant spaces by maintaining a smaller corewall thickness, floor slab contribution to lateral load resistance as a moment frame was considered for both strength and stiffness design. Its contribution by relative stiffness is about 25% of story shear at the high-rise floors and between 10% and 20% at the low and mid-rise floors. Thus, the entire lateral load resisting system did not qualify as a dual system with a modification factor, R=6. In order to qualify as a dual system, the moment frame must be designed to resist at least 25% of the lateral base shear forces independently. It was found that if the lateral system were designed as a dual system, it would require high flexural and shear reinforcement in the slab around the column and the wall areas while reduction of the core wall thickness would be minimal due to tension forces. The benefit did not justify the cost. Therefore, the entire lateral load resisting system, both the core wall and the equivalent slab moment frame, was analyzed together and designed for seismic forces as if a corewall system with a modification factor, R=4.

These floor slab moment frames were designed as ordinary moment frames. For example, two-way slab shear strength was based on the code stipulated combination of forces, not necessarily twice the seismic forces as for an intermediate moment frame. Throughout the project, slab shear reinforcement around the column was minimal. However, it was decided to provide intermediate moment frame details for floor slabs and columns in order to achieve some level of ductility in such tall towers.

5. Link Beam Design

A completely enclosed core wall is much stiffer than a wall with openings. However, if the core walls are connected with stiff members across the openings, the entire core wall would act as a tube member rather than individual shear wall pieces. The connecting members are often called link beams or coupling beams. Three different types of link beams have been utilized for The # Star City project.

Link beams that are located inside the core or across door openings where shear forces induced by the lateral loads are not excessive have been designed as reinforced concrete beams with both flexural and shear reinforcement.

Link beams that are located across the elevator lobby entrances at the middle of north and south of the core were to be designed to accommodate four 215 mm (8.5 in.) ~ 267 mm (10.5 in.) diameter and a 230 mm (9 in.) by 250 mm (10 in.) web openings, while the total depth of the beams was kept at 790 mm (31.5 in.) as shown in Figures 6 and 7.

At the high-rise floors, the shear forces in the link beams are not excessive thus the link beams have been designed as reinforced concrete beams with heavy reinforcement around and between the web openings as shown in Figure 6.

At the lower levels, the shear forces are relatively large. In response to such large shear and flexural forces induced in the link beams across a lobby opening in the core walls, composite beams (structural steel wide flange members encased in concrete) were used in lieu of conventional reinforced concrete (RC) beams. The flanges have been designed to resist the moment induced by the shear in the link beam while the web has been designed to resist the shear forces while accounting for large web penetrations.

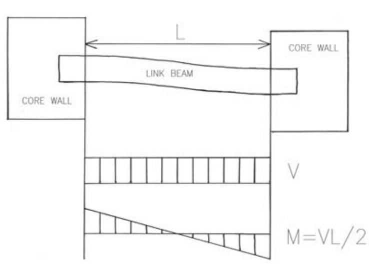


Fig. 5. Link Beam Forces

A unique approach was introduced for the steel link beam embeddment design due to the relatively large shear forces induced by the lateral forces between the core walls. With such large end forces, the conventional embeddment approach caused lots of detailing problems associated with enlarged concrete bearing and the strut-and-tie shear check of the wall between floors for both horizontal and vertical directions. As shown in Figure 7, the web of the link beam is extended 550 mm (22 in.) inside the corewall and only the beam flanges are extended beyond the pullout failure envelope.

End shear and moment forces in the steel link beam have been de-coupled. The vertical shear forces are transferred to or from the concrete wall below and above the link beam within the 550 mm (22 in.) of the web plate. End moments of the beam have been coupled to the tension and compression forces in the flanges. Axial forces in the flanges are in turn transferred to or from the core wall through the shear studs and the end plates.

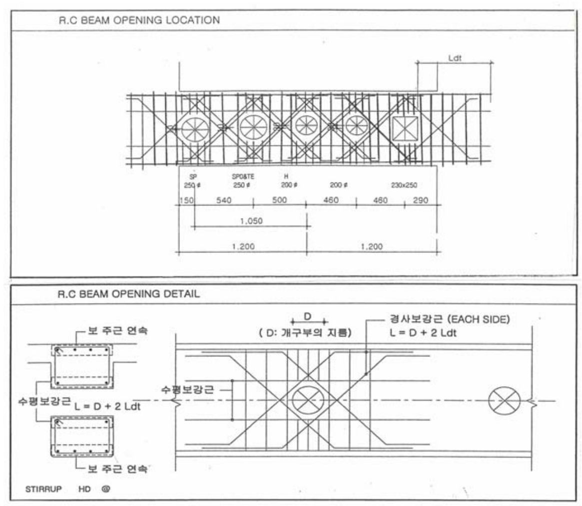


Fig. 6. RC Link Beam with Web Openings

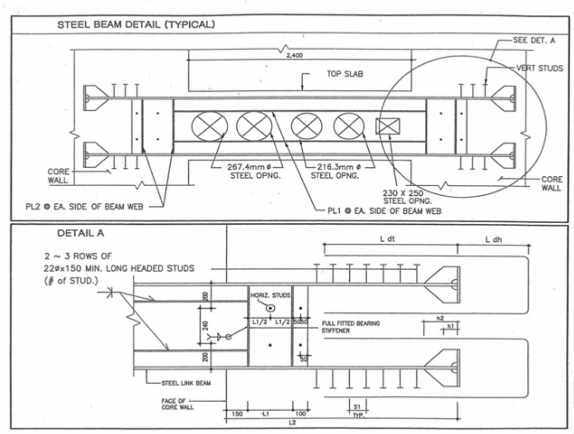


Fig. 7. Steel Link Beam with Web Openings

6. Differential Column Shortening

In such tall towers, corewalls and adjacent columns have different stress levels, causing differential shortening through elastic and inelastic phenomena. Differential shortening between columns, and the between columns and the corewall, produced additional moments in the slabs.

Both elastic and inelastic shortening was estimated using an in-house program, reflecting proposed member properties and the construction schedule provided by POSCO E&C at the end of DD phase. Concrete strength, member size, rebar %, loads, construction schedule, humidity, age (time) were controlling factors for the column shortening. Depending on the spacing between columns and corewall, concrete strength, and thickness of slab, different end moments in the slab have been found. Therefore, all factors have been considered for the project to minimize the differential shortening impact.

Since shortening will progress slowly for years, there will be relaxation of the induced moments in the slabs. Therefore, 50% reduction in the long-term column shortening impact has been assumed for the slab reinforcement design.

7. Foundation

The soil/rock conditions at the expected foundation level throughout the site are poor and heterogeneous. Soil bearing capacities directly under the tower footprints varied from 110 to 180 T/m^2. Therefore, it was planned at a very early stage that the tallest tower be located over the relatively strongest rock. Please see the table below for the summary of foundation system and soil bearing capacities of each tower

Due to the poor rock bearing capacity and variation of rock quality, a mat foundation system was utilized to transfer the tower weight and lateral loads to the soil without causing differential settlement of the foundation. Mat foundation thickness was controlled by the two-way shear capacity needed.

Table 6. Summary of Foundation

Note: "-18.2 m" indicates an elevation of 18.2m below grade.

	Allowable Rock Bearing capacities (T / m^2)	Subgrade Reaction Kv (T / m^3)	Mat Thickness (m)	Top of Mat Elevation
Tower A (60-Story)	180	25,000	3 m	–18.2 m
Tower B (40-Story)	110	20,000	2.5 m	-19.2 m
Tower C (50-Story)	160	20,000	3 m	-16.2 m
Tower D (45-Story)	150	30,000	2.5 m	-16.7 m
Podium (B3)	35 - 50	-	Up to 1.2 m	-13.0 m

Each tower mat was cast in one day using f'c = 300 kgf/cm^2 (4.26 ksi) concrete. In an effort to lower the heat of hydration of the massive concrete mat, flyash was used as a partial cement replacement in the concrete mixture. Other advantages of using flyash including improvement of both performance and quality of concrete by reducing water demand, segregation, bleeding, permeability, corrosion of reinforcing steel, and alkali aggregate reaction were also realized. Temperature of concrete was regularly monitored in the field during the curing periods.

Fig. 8. Preparation of Mat Construction

Fig. 9. Formwork of Mat Foundation

8. System Consideration for High-Rise Tower

Many factors, including those affecting building efficiency, project cost, construction schedule, and safety were considered in the process of structural system selection. The following is the checklist used for The # Star City project.

1. Establish design criteria
   - based on the local building code for live loads
   - wind and seismic loads
   - based on the local material design codes such as ACI 318-99 or 318-02 for concrete, ASD or LRFD 3rd edition for steel
   - based on a parallel check with internationally accepted building codes
2. Determine locally practical structural materials
   - availability (different grades of steel, different

strengths of concrete)
- reliability (quality control in cements, aggregates, admixtures, batching controls and mixing of high-strength concrete mixtures)
- constructibility (ability to erect large, heavy steel members)
- relative cost

3. Floor-to-floor heights
   - increased height has major effect on wind loads and especially on story drift (magnitude of sidesway)
   - non-structural components also affected (cladding area, riser lengths, elevators, stairs)

Fig. 10. Recent Tower Construction Pictures

4. Establish human comfort criteria under wind-induced building motions
   - long-term residential, short-term residential (hotel), and office use have different criteria
   - cultural response to severe wind conditions
5. Establish building deflection limits for non-structural elements such as
   - exterior curtain walls -- can affect mullion widths, joint sizes between panelized units, and support details
   - interior partition walls
6. Determine governing stiffness requirements based on wind strength, wind comfort, non-structural or seismic requirements
7. Foundation issues
   - concentrated building weight affecting strength and settlement requirements
   - possible net uplift forces and overturning
   - dewatering for deep basements
8. Differential column shortening and column cambering

9. Conclusion

This paper presents an outline of the structural system and major concerns for the tallest high-rise concrete residential tower construction in Korea. This project is currently under construction. The team is targeting the top-out date of the towers to be in the second quarter of 2005, interior finish by the third quarter of 2006, and the tenant move-in by July of 2007.

Extensive collaboration and co-operation between the design teams in Korea and the United States was essential to successfully implementing the mega-high-rise residential towers' design and construction.

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