Displacement-Based Seismic Assessment of Moment Resisting Frames with Blind Bolted Connections

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ABSTRACT:

Although concrete-filled tubular columns have high load capacity with small cross-sectional area, favorable ductility and high energy absorption capacity, they are not popular in Australia due to the perceived difficulty of connecting them to beams. Due to the lack of access inside the tube in the beam-column connections, it is difficult to install standard bolts. Recently, innovative blind bolted connections, ranging from semi-rigid to rigid, have been developed to avoid welding at site.

In this paper the results of an investigation into the adequacy of newly proposed lateral force-resisting systems is presented. The systems consist of moment resisting frames with concrete filled steel tubular columns and blind bolted connections between these columns and steel Universal Beam sections that are composite with a concrete slab. They include a combination of perimeter and internal frames. The framing systems have been designed using Australian and European standards as appropriate. The seismic design has been carried out assuming that the buildings are situated in Melbourne and using the Australian Earthquake Loading Standard (AS 1170.4:2007) in order to be consistent with the usual design practice in Australia, i.e. a force-based design approach in which the intention is to satisfy the performance objective of life safety in a 500 year return period earthquake design level event. A capacity spectrum approach has been used to perform the seismic assessments to determine the actual performance at key earthquake levels such as the 500 year and 2500 year return period events. Nonlinear pushover analyses have also been performed on the structures. Two different methods have been explored to improve the ductility of the connections and to enhance the overall displacement capacity of the buildings.

Keywords: Blind bolt, Concrete-filled steel tube, Moment resisting frame, Seismic assessment, DBA

1. INTRODUCTION:

Concrete members have a high compressive strength and steel members have a high tensile strength and ductility. Concrete filled steel tubes (CFST) combine the beneficial qualities of concrete and steel in one member. CFST columns have a large number of advantages over other columns. Those structural, economical and architectural advantages make them popular in building construction.

A large floor span which provide open column-free spaces is always preferred in buildings. For this purpose, moment resisting frames using CFSTs as the columns have been popular in Japan, China and the USA for many years. Those framing systems use extensive welding in the beam-column connections, but in Australia field welding is not favoured due to cost and quality control issues. Shop welding and field bolting is the common practice. Hence, the structural systems here are to be developed using blind bolted connections that have an adequate strength and stiffness. A number of blind bolted connections have been proposed but there are no guidelines available for designing such blind bolted moment resisting connections.

The type of blind bolt used in this study was the Ajax ONESIDE. It was developed in Australia (Ajax Engineered Fasteners 2005) and can be installed from the external side of the column using a simple installation tool. This reduces the labour requirements and reduces the cost. Furthermore, the full strength of the bolt can be achieved, unlike other blind bolts. In this research, a modified innovative blind bolt will be used. The components of the blind bolts (bolt, screw and washer) are made of property class 8.8 steel with nominal yield stress of 660MPa and tensile strength of 830MPa.

Is this paper a moment resisting framing system is designed which incorporates blind bolted beam-column connections. The case study building has large floor mass and long floor span and hence a high fundamental period. When it is subjected to lateral earthquake forces, the stability need to be considered. AS1170.4 (Australian Standard 2007) defines a stability factor (θ) to check the stability of structures. A limiting value is specified in the standard and is same for all type of building structures. Detailed discussion is presented here regarding the stability issue in such structures. Also the adequacy of the proposed structural system comprising of blind bolted connections is investigated under seismic demands corresponding to both 500 and 2500 years return period earthquakes.

2. DESIGN OF BUILDING:

2.1 Building Description

A typical five storey office building is selected in this study. Figure 1 shows the plan, elevation and section view of the moment resisting frame (MRF) of the building. Connections of the MRF are blind bolted connections except those with a cross (x) at the beam end for which the connection is a pinned connection. The study building is located in Melbourne in site soil class B.

The columns of the building are made of Concrete Filled Steel Tubes (CFST) and the beams are Universal Beams (UB) composite with a 140mm slab. Normal weight concrete of 50MPa and light weight concrete of 32MPa are used in column and slab respectively.

2.2 Loadings

The building is subjected to Dead Load, Live Load, Wind Load and Seismic Load. All of the loadings were calculated based on relevant Australian standards. The equivalent static method specified in Chapter 6 of AS1170.4 was used to calculate the seismic forces. The method used to calculate these forces will be discussed further in section 3.1. Since the building is symmetrical in both horizontal axes, no eccentricity is expected in normal operation condition, but some accidental eccentricity could occur. To account for this an accidental eccentricity of 10% of building width was considered, as specified in code.

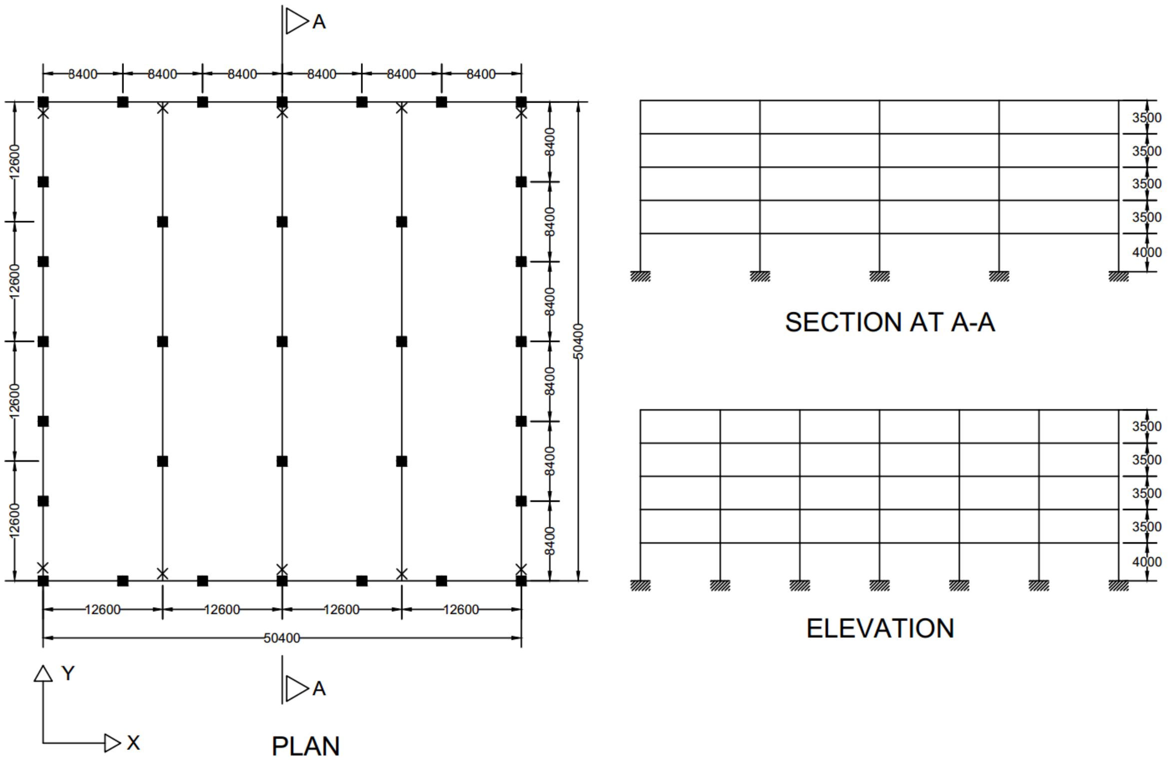

Figure 1: Plan and elevation of building (All dimensions are in mm)

There is no specific method given in AS1170.4 to calculate the period for composite structures. In Cl 6.2.3 of AS1170.4 a formula is given for concrete and steel structures only. Since composite structures are more flexible than concrete ones, the formula given for steel structures gives more accurate value than that for concrete structures. Therefore, to calculate the fundamental period, Eq. 6.2(7) from AS1170.4 was used with k_t=0.11, which gives T_1=1.2 Sec.

2.3 Analysis and Design of Building

A finite element program called ETABS 2013 (Computers and Structures, Inc. 2013) was used to analyse the building. Second order analysis was performed to consider the P-Delta effect, and that is discussed further in section 3.2.

Since there is no Australian standard to design the continuous composite beam and CFST columns, Eurocode (CEN 2004) was used in designing composite beams and columns. Col 3 of the Table 1 shows the designed section sizes of the beams.

The moment capacity of composite beams in the region of hogging bending moment is very high as compared to that in the region of sagging bending moment. But the bending moment obtained from elastic analysis is usually higher near the support than at the mid-span of the beam. So, to take the full advantage of moment capacity of the composite beam along its full length, redistribution of moment from the hogging region to the sagging region was carried out when designing the building. The member sizes were governed by the negative moment at the support due to the gravity load combination 1.2G+1.5Q i.e., the earthquake and gravity combination of G+0.3Q+EQ did not govern.

Table 1: Designed sections and load combinations

Beam	**Span (m)**	**Sections**		**Governing load combination**
		Considering –ve rebar	**Without considering –ve rebar**	
(Col 1)	(Col 2)	(Col 3)	(Col 4)	(Col 5)
Perimeter Beam (Along Y-Direction)	8.4	360UB56.7	460UB74.6	1.2G+1.5Q
Perimeter Beam (Along X-Direction)	8.4	410UB53.7	460UB74.6	1.2G+1.5Q
Interior Beam	12.6	610UB125	610UB125	1.2G+1.5Q
Secondary Beam	12.6	310UB40.4	310UB40.4	1.2G+1.5Q

3. DISCUSSION ON DESIGN METHOD:

3.1 Calculation of Base Shear and Inter-storey Drift

As discussed in Section 2.2, from AS1170.4 the fundamental period of the building is found to be 1.2 sec. However, from the modal analysis, the fundamental period was obtained as 3.1 sec in the X-direction and 2.1 sec in the Y direction, which are both significantly greater than those obtained from the standard. If the lateral forces corresponding to this period from AS1170.4 are applied to the bare frame and the results of this analysis are used to calculate the drift, the drifts will be overestimated (O'Brien et al. 2010). Therefore it is reasonable to calculate drift using lateral forces that are consistent with the computed period from modal analysis. The American standard (ASCE/SEI 7-05 2006) allows the computed period to be used. The Australian standard for earthquake design (AS1170.4) also allows the computed period to be used but the base shear should not be less than 80% of the value obtained with the period calculated using eq 6.2(7) from the code i.e., T_1=1.2 seconds. Figure 2 shows the acceleration response spectra defined in AS1170.4. The dashed line in the figure is scaled down from the solid line by the factor μ/S_p and relationships between different points are shown in the figure. For example, the seismic force corresponding to A_3 is 80% of that corresponding to A_2 and the force at point A_1 is S_p/μ time that of A_2.

3.2 Comparison between First and Second Order Analysis

AS1170.4 allows first order analysis to be used but later it requires the stability factor (θ) to be checked. All the forces and displacements from the first order analysis need to be adjusted by scaling factors if the value of θ is between 0.1 and 0.2. For the stability of the building, the θ value must not be greater than 0.2 in any storey no matter which type of building it is. If the θ value is greater than 0.2, the building is said to be unstable and needs to be redesigned.

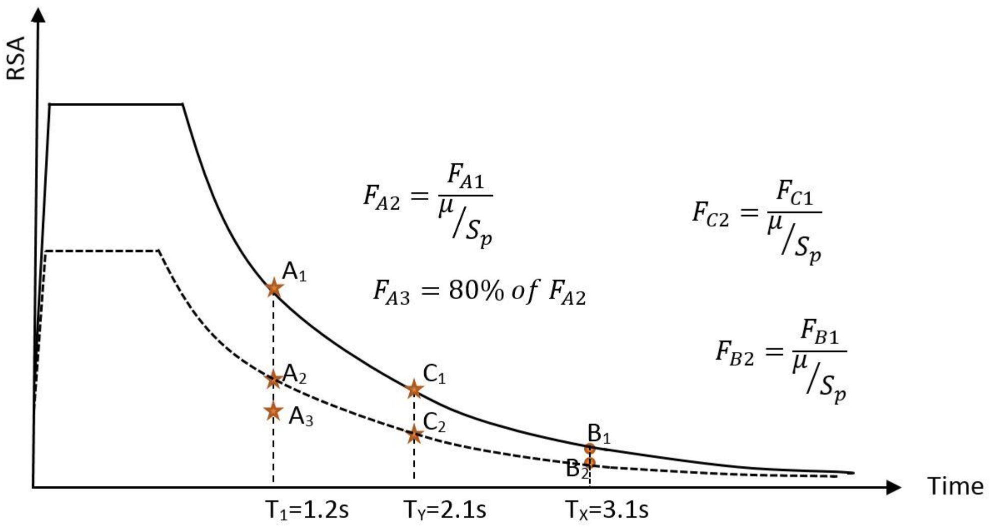

Figure 2: Acceleration Response Spectra where T_1 is from AS 1170.4:2007, T_X and T_Y are modal periods in the X and Y directions respectively (figure not to scale).

Table 2: Base shear corresponding to different points shown in Figure 2

Base Shear at (kN)						
A_1	A_2	A_3	B_1	B_2	C_1	C_2
3980	1530	1230	740	280	1560	600

In the study building, some of the calculated values of θ from the first order analysis were found to be greater than 0.2 in the weaker direction (X-direction). To bring this θ value down to its limiting value, the beam sections would need to be increased significantly and this would increase the mass of the structure and thus the cost. The stability of the building was checked using the American code (ASCE/SEI 7-05 2006) and this suggested that the building is stable (i.e., the θ value calculated in accordance with that standard was below the limit specified).

AS1170.4:2007 allows the use of a second order analysis instead of performing a first order analysis and then using θ factors, and so a second order analysis was carried out for the case study building. The second order analysis needs to take into account the actual lateral displacements of the structure (AS 1170.4 Commentary 2007). The structure is analysed in the displaced position to calculate the increased actions and displacements due to P-Delta effects. Several iterations may be needed before the displacements stabilise. It is clearly important to accurately estimate the first order displacements if this process is to give valid results.

The lateral displacements found in a first-order analysis by applying the forces corresponding to the response acceleration A_3 to the bare frame would over-estimate the real displacements of the frame in a 500 year return period earthquake, especially given that AS1170.4 requires that these displacements be increased by a factor of μ/S_p in accordance with the equal displacement principle. As pointed out by O'Brien et al. (2010) realistic estimates of the displacement of an equivalent SDOF structure representing the frame can be found simply by using the elastic displacement spectra at the periods calculated using modal analysis. Given that the building is sited on rock the maximum displacement of the SDOF structure would be 26mm in a 500 year return period event in accordance with AS1170.4. This is shown in Figure 3. Hence a better estimate of the first order displacements would be given by calculating the seismic forces for the X and Y direction corresponding to the acceleration B_1 and C_1 respectively on the elastic acceleration response spectra, as shown in Figure 2.

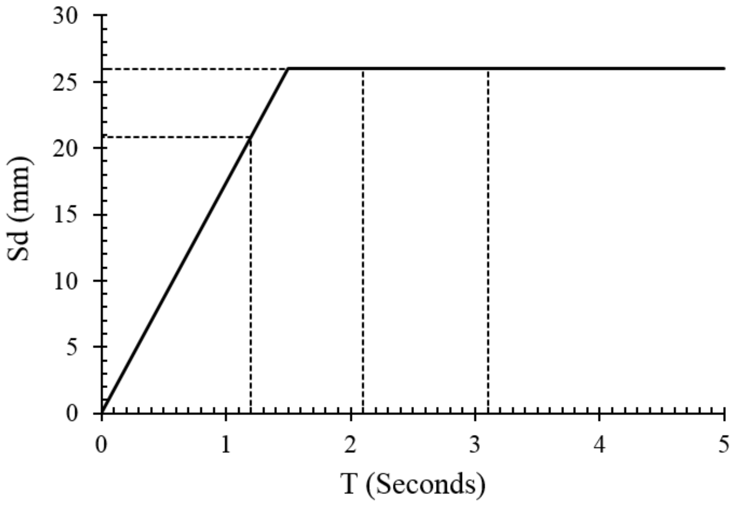

Figure 3: Elastic displacement response spectra

To simplify matters the following procedure is used. In Table 2 the base shear corresponding to the different acceleration response points on Figure 2 are given. In the X direction the base shear from B_1 is less than that from A_3, so the base shear at A_3, i.e., 1230 kN, is used in the second order analysis. As mentioned previously A_3 is the minimum acceleration response that is allowed to be used in accordance with AS1170.4. In the Y direction the base shear from C_1 is greater than that from A_3. Hence the base shear at C_1, i.e., 1560 kN, is used to calculate the storey forces to be used in the second order analysis in the Y direction. In effect the frame is designed using the second order actions and displacements obtained by applying seismic forces corresponding to the spectral acceleration at A_3 in the X direction and at C_1 in the Y direction. The calculated displacements are equal to or greater than those corresponding to the elastic response of the structure at the modal periods of 3.1 seconds and 2.1 seconds, so it is not necessary to factor them by the μ/S_p factor.

Results from first and second order analyses are presented in Table 3 which compares the displacement at each floor level. Storey displacements from the second order analysis are about 20% and 10% greater than that from the first order analysis in the X and Y direction respectively which is normal in such buildings (FEMA 451 2006). The increase in moment also followed the same trend. The inter-storey drifts have been calculated using the displacements from the second order analysis and they are all less than 1.5% as required in AS1170.4.

Table 3: Comparison of displacement of first and second order analysis

Storey	**Storey Height (mm)**	**Displacement in X-Direction**			**Displacement in Y-Direction**		
		1st Order (mm)	**2nd Order (mm)**	**Change (%)**	**1st Order (mm)**	**2nd Order (mm)**	**Change (%)**
Story5	3500	66.6	81.8	22.8 ↑	47.1	52.2	10.8↑
Story4	3500	54.7	67.5	23.4 ↑	40.1	44.7	11.5↑
Story3	3500	40.0	49.6	24.0 ↑	30.5	34.1	11.8↑
Story2	3500	23.7	29.4	24.1 ↑	18.9	21.2	12.2↑
Story1	4000	8.7	10.6	21.8 ↑	7.4	8.3	12.2↑

4. PLASTIC BEHAVIOUR OF BUILDING:

The office building used in this study has CFST columns and a composite slab system with blind-bolted moment resisting connections. For buildings like this, with large floor spans, the beam sizes are large due to heavy gravity loads. Also the CFST columns have a very large capacity such that they are less likely to go into the plastic range adjacent to the connection to beams. The strength hierarchy at the joints are such that it makes sense to design the connection to be the weak ductile link. In the proposed blind-bolted connections, the forces in the bolts are limited to 60% of their capacity, even under severe earthquakes.

Agheshlui (2014) proposed a T-stub connection as shown in Figure 4 where the flange of T-stub will yield to dissipate energy in the connection. The T-stub needed to be designed in such a way that the flange would yield in flexure when the forces in the blind-bolts reached 60% of their capacity.

Clifton (2005) proposed an asymmetrical friction connection called a Sliding-Hinge Joints (SHJ). In this connection the beam is connected to the column through top and bottom flange plates. The top flange plate is pinned to the column at the top flange of the beam and this top flange connection acts as a point of rotation for the connection. There are slotted holes in the bottom flange plate and normal sized holes in the two shims and the beam flange as shown in Figure 5. There are fully tensioned bolts passing through the connecting plate, the beam flange and the two shims. When the friction capacity is exceeded there is sliding at the shim-flange plate surfaces. The same principle can be used with blind bolted connections. The top T-Stub is connected to the CFST with blind-bolts and is designed to remain strong and stiff. It acts as a centre of rotation. The web of the bottom T-Stub with slotted holes allows the beam to slide against the beam flange. The addition of a thin shim plate between the T-Stub and beam flange would increase the friction coefficient and thus the frictional resistance. The connections need to be designed in such a way that no sliding occurs under a 500 year return period event and after that it starts sliding.

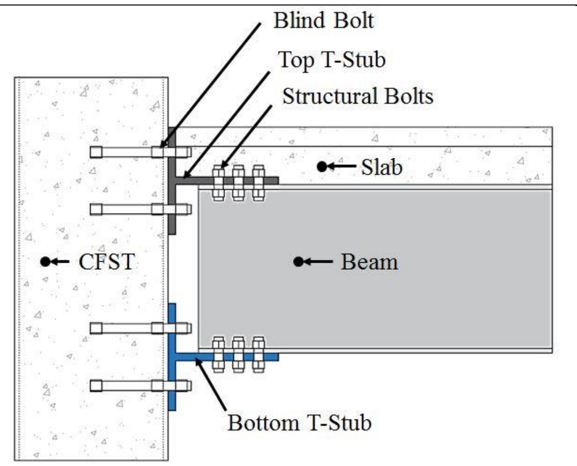

Figure 4: T-stub connection to CFST

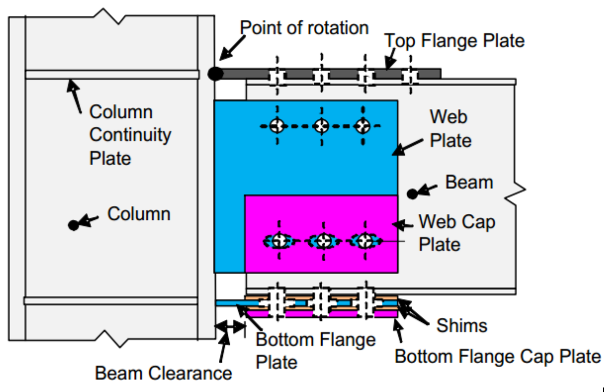

Figure 5: Sliding hinge joint (MacRae et al. 2010)

If the beam rotates about the top corner of beam, unlike the T-Stub yielding case, the reinforcement within the slab will not be activated. Thus, the capacity of composite beam near the connection (in the hogging region) decreases and larger sections are required to meet the demand. Col 4 of Table 1 shows the section sizes required for such case.

5. SEISMIC ASSESSMENT:

The capacity spectrum method is used to assess the behaviour of the building under a 500 and 2500 year return period earthquake. The ADRS diagram shown in Figure 6 was prepared based on Lam and Wilson (2008). The pushover curve obtained from the nonlinear static analysis considering the P-Delta effects is also shown in the figure. As can be seen from the figure, the building can tolerate the lateral displacements obtained from 2500 year return period earthquakes.

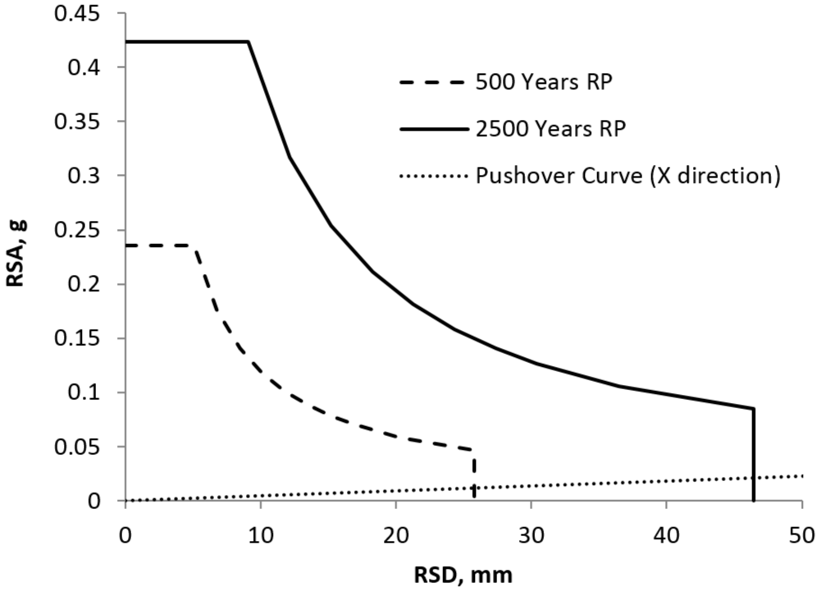

Figure 6: Capacity spectrum for 500 and 2500 years RP earthquakes for site soil class B

6. CONCLUSIONS:

Based on the numerical work reported in this paper, the following conclusions can be drawn:

- A case study building consisting of moment resisting frame only as the lateral force resisting system (i.e., without the need for walls) has been successfully designed for five storey office building using CFST columns, steel beams and blind bolted connections.
- The stability factor specified in the design standard AS1170.4 sometimes gives misleading information. In this case study of a five storey office building, the θ value from first order analysis calculated in accordance with AS1170.4 was found to be greater than limiting value of 0.2, suggesting that the structure was unstable. However, from second order analysis, it was found to be stable.
- Two different methods have been explored to improve the ductility of the connections and to enhance the overall displacement capacity of the buildings.
- The performance of the building was evaluated based on the capacity spectrum method and it was found that the building can tolerate the spectral displacement from a 2500 year RP earthquake. In fact the building remains elastic, when it is situated on a site of class B.

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