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Behaviour of Strengthened Reinforced Concrete Beam-Column Joints

Comportement des noeuds de cadres en béton armé après consolidation

Verhalten von Stahlbetonrahmenknoten nach Verstärkung

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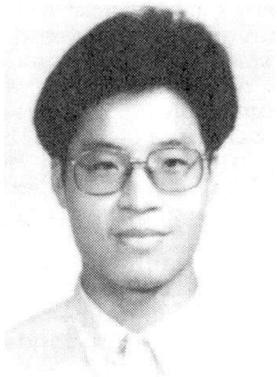


Longmei Shentu, born in 1965, received his Master's degree of engineering at Tongji University. For six years, he was involved not only in the design of reinforced concrete structures, but in research and design of projects related to structural strengthening as well. His research interests also include finite element analysis.

SUMMARY

Based on the simulation of existing reinforced concrete beam-column joints in which seismic loads have not been considered in design or whose seismic strength is not adequate, this paper presents an experimental investigation of different strengthening methods with externally glued steel. Using the pseudo-dynamic test method, two sets of specimens including a total of seven specimens are presented. Strengthening methods were developed to improve the shearing strength at the cores, while two further methods are presented to strengthen the positive flexural strength at the beam ends. The experimental results show that joints properly strengthened with glued steel can exhibit greatly enhanced seismic performance.

RÉSUMÉ

L'auteur présente différentes méthodes de renforcement par collage de tôles d'acier à l'extérieur des noeuds de poutres-cadres en béton armé, qui ont été à l'origine insuffisam-ment dimensionnés sous charge sismique. Il examine les essais pseudodynamiques effectués sur sept échantillons. Il expose des procédés de renforcement pour augmenter la résistance au cisaillement des noeuds, ainsi que pour augmenter la résistance à la flexion des jonctions montants-traverses de ces cadres, cette consolidation étant réalisée à l'aide de tôles d'acier collées. Les résultats expérimentaux montrent que les noeuds correctement renforcés offrent des performances sismiques nettement plus élevées.

ZUSAMMENFASSUNG

Der Beitrag behandelt unterschiedliche Verstärkungsmethoden für Stahlbetonrahmenknoten, bei deren Bemessung Erdbebenbelastung unzureichend berücksichtigt wurde. In pseudodynamischen Versuchen wurden insgesamt sieben Prüfkörper in zwei Serien untersucht. Drei Verstärkungsmethoden wurden für die Erhöhung der Schubtragfähigkeit im Knotenkern entwickelt, zwei weitere für die Erhöhung der Biegetragfähigkeit der Riegelanschlüsse. Verwendet werden extern aufgeklebte Stahlbleche. Die Versuchsresultate belegen, dass Knoten mit richtig angebrachten Stahlverstärkungen ein deutlich erhöhtes seismisches Leistungsvermögen aufweisen.

1. INTRODUCTION

With advantages such as fast construction, less additional weight, and only minor changes in the shapes of the beams, the strengthening of reinforced concrete beams with externally glued steel plates has been successfully utilized in many projects.

Seismic behavior of reinforced concrete beam-column joints has been broadly studied, which has greatly improved the seismic design in the RC structures. In existing buildings, however,there are many cases in which the structures were designed without considering the seismic loads or the existing seismic strengths will not be strong enough to withstand newly predicted major seismic events. In some cities such as Shanghai in China, the new provisions of seismic design demand higher seismic strengths than were standard, and many important buildings have to be strengthened to meet the new provisions. In these cases,the cores in a RC frame usually could bear only small shearing forces and the beam ends could hardly bear any positive moments although they usually could bear negative ones. It would be practical to employ the strengthening method of externally glued steel plates to improve the seismic performance of these structures.

The cyclic behavior of the bond between concrete and steel plate had been tested[1] to be sure that the glued steel plate could work well with the concrete beam under cyclic loads. Subsequently seven specimens of beam-column joints were tested, using five different strengthening methods. The experimental results of these tests are presented in this paper.

2. TEST SPECIMENS AND EXPERIMENTAL MODEL

Though in practice it is usual that both cores and beam ends need to be strengthened, separated strengthening methods were adopted in these tests, in which only the cores(SET I) or the beam ends(SET II) were strengthened to clearly investigate the difference between the strengthening methods. The seven specimens had the same size having the columns 1,910mm long and cross sections 200mm wide and 250mm deep, and the beams also 1,910mm long, which included the column depth, and their cross sections 120mm wide and 250mm deep.

In SET I, four specimens, which are identified as J1,J1G1,J1G2 and J1G3 were tested. Each specimen had longitudinal steel reinforcement consisting of one 16mm-diameter deformed bar at each corner of the beam and the column while the shear reinforcement consisted of 6mm-diameter stirrups at 100mm spacing, except the joint core where no transverse bars were provided. Therefore, the seismic load that the column and the beam could bear was much bigger than that of the core. If there are no longitudinal beams between reinforced concrete frames, it is possible to glue steel plates along both lateral surfaces of the beams such as J1G1 shown in Fig.1a, in which the glued steel plates will act just as the transverse bars within the cores. However, longitudinal beams are usually provided to strengthen the stability of frames along the longitudinal direction and make the strengthening method of J1G1 impossible, thus a new strengthening method was developed in J1G2, in which four pieces of 40x4mm angle steel were glued at the column corners and they were connected by closed ties of steel plates at 100mm spacing, except the area of the beam depth. In order to greatly confine the core, another four pieces of the same size angle steel were provided to strengthen the lateral rigidity of the former angle steel as shown in Fig 1.b. The difficulty with this strengthening method is that the slab at column corners must be punched to let the angle steel go through. To avoid this inconvenience in construction, the strengthening method J1G3 shown in Fig 1.c was developed. A comparison with J1G2, in which the only difference was that the angle steel in J1G3 was cut in the place of the imaginary slab, could now be drawn.

In SET II, three specimens identified as J2, J2G1 and J2G2 were tested. The column of every specimen in this set was provided with four 16mm-diameter longitudinal deformed bars, while the beam had two 16mm-diameter bars on top and two 12mm-diameter plain steel bars at the bottom. The transverse steel bars in both the column and the beam consisted of 6mm-diameter stirrups at the space of 100mm while five 10mm-diameter plain steel stirrups were provided within the core. Since the purpose in this set was to find a way of improving the flexural strength at beam ends during seismic events, the flexural strength at beam ends was the only item that needed to be strengthened. If there are longitudinal beams, the

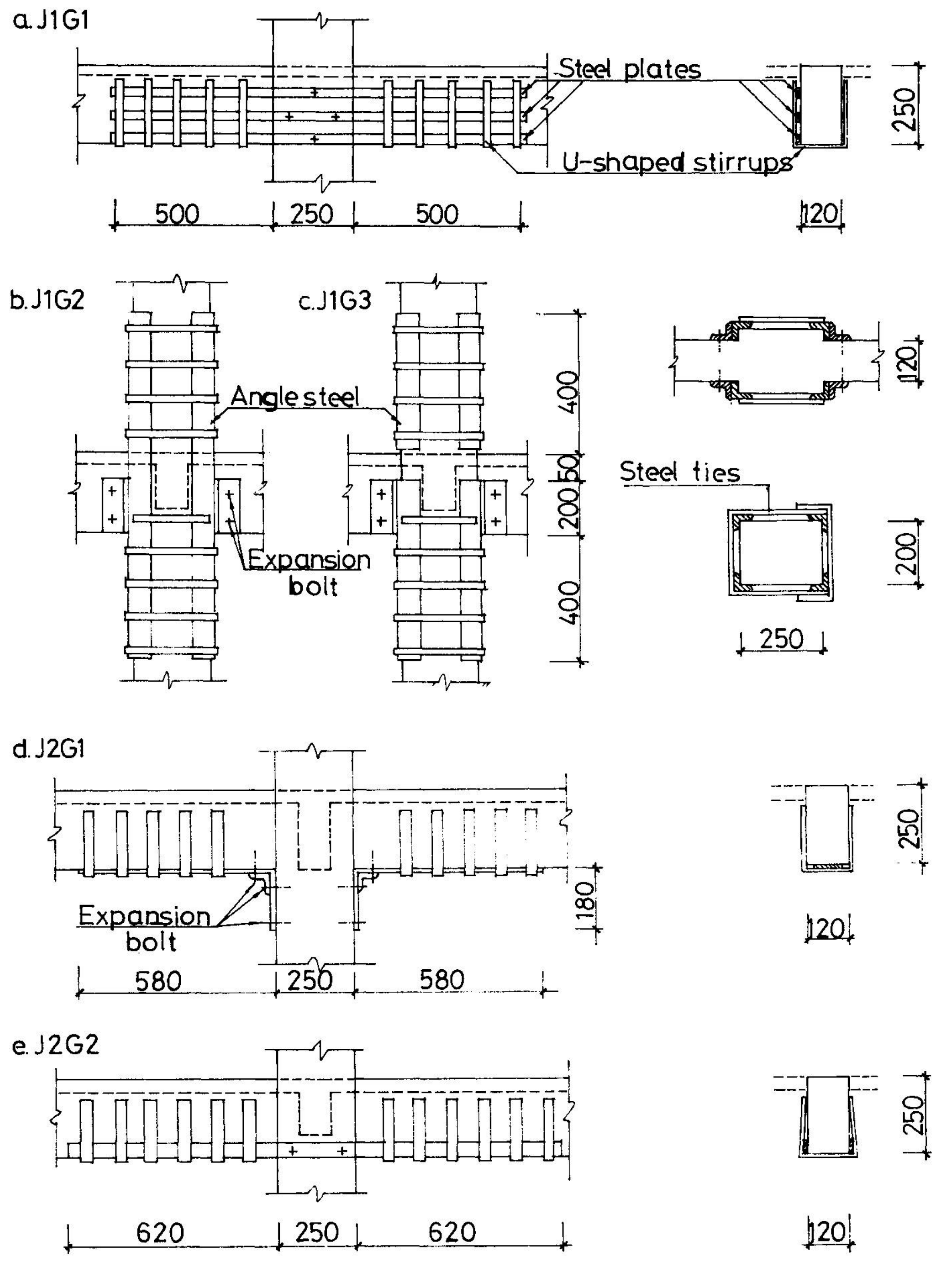


Fig. 1 Strengthening methods

depth of which is less than that of frame beams, it is possible to glue steel plates on the lateral surfaces of the frame beams as shown in Fig 1.e (J2G2). However, when the depth of the longitudinal beams is the same as or very near to that of the frame beams, it is impossible for the joints to be strengthened as in J2G2. Another strengthening method shown in Fig 1.d (J2G1) was developed where the steel plate was glued on bottom surface of the beam and bent vertically when it approached the column, and a piece of angle steel was used to strengthen the anchorage between the plate and the column. Both J1 and J2 which remained unstrengthened were used for comparison. And the properties of the materials used in the experiments are shown in Table 1.

Since these tests concentrated on the improvement of the shearing strength at the cores and the flexural strength at the beam ends under seismic loads, the effect caused by the lateral displacement at tops of the columns was relatively minor. The experimental model used is shown in Fig.2, in which the cyclic loads were antisymmetrically acted at the two free ends of the beam, while a constant vertical load was acted at the column ends where the lateral displacements were restrained.

Materials		Yield strengths (MPa)	Ultimate Strengths (MPa)
Steel bars	$\phi 6$	218	394
	$\phi 10$	293	461
	$\phi 12$	238	392
	$\phi 16$	361	568
Steel plate	t=2mm	308	430
	t=3mm	256	335
Concrete	f_c=18.54 MPa		

Table 1. Properties of materials

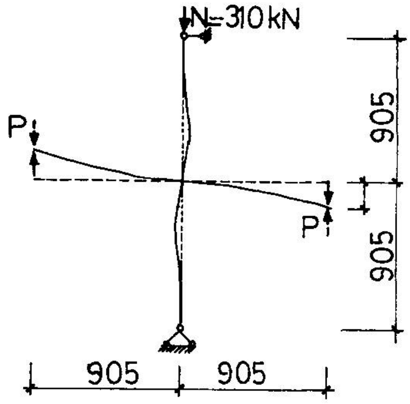


Fig.2. Experimental model

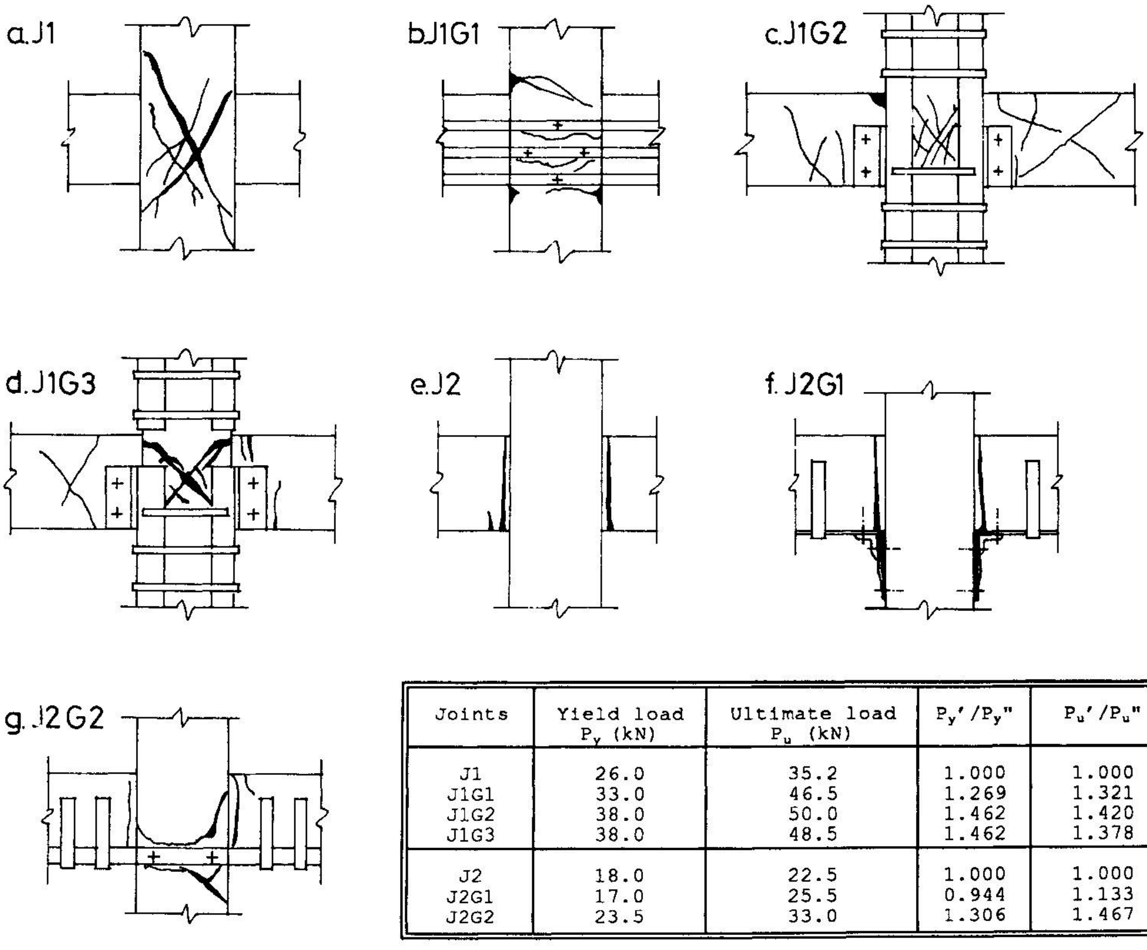


Fig.3. Failure of specimens

Joints	Yield load P_y (kN)	Ultimate load P_u (kN)	P_y'/P_y''	P_u'/P_u''
J1	26.0	35.2	1.000	1.000
J1G1	33.0	46.5	1.269	1.321
J1G2	38.0	50.0	1.462	1.420
J1G3	38.0	48.5	1.462	1.378
J2	18.0	22.5	1.000	1.000
J2G1	17.0	25.5	0.944	1.133
J2G2	23.5	33.0	1.306	1.467

P_y', P_u':strengthened; P_y'', P_u'':unstrengthened

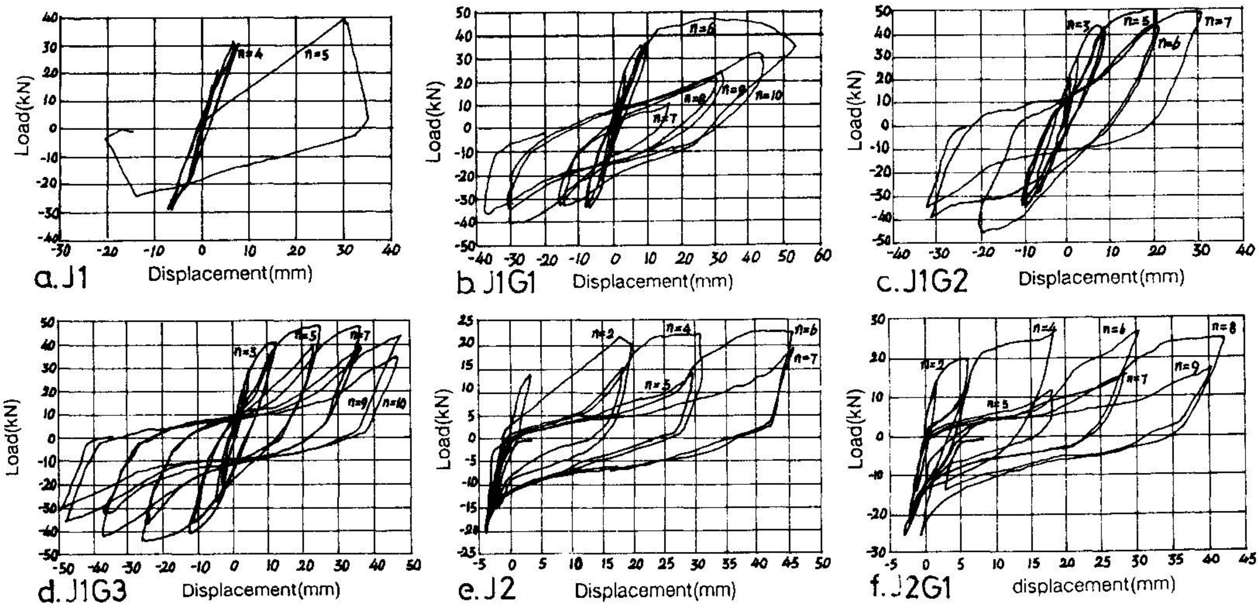
Table 2. Yield and ultimate loads

3. EXPERIMENTAL RESULTS

3.1 General description

The forms after the specimens collapsed are shown in Fig.3. In SET I, abrupt shearing failure took place at the core of J1, where the cracks were wide and long, and some concrete was peeled off. The plastic hinge in J1G1, however, was presented at the column section while only some small horizontal cracks developed at the core, which means the shearing strength at the core had been greatly enhanced. In J1G2, because of the strengthening by the angle steel at the column corners, the shearing strength at the core and the flexural compressive strength of the column had been greatly improved, thus,the plastic hinge occured at one of the beam ends although many shearing cracks also developed at the core as shown in Fig 3.c. Fig 3.d presents the failure of J1G3 in which two obvious diagonal cracks were developed to indicate the shearing failure at the core contrasting with the plastic hinge at the beam end in J1G2.

In SET II, plastic hinges developed at the beam ends in each specimen as shown in Fig.3. In J2, the displacement in positive direction was very large when its load was small because of the very small positive flexural strength at the beam ends. J2G1 had exhibited almost the same behavior of deformation as that of J2 though it had been strengthened with the glued steel plates. This is because that the anchorage between the steel plates and the column failed after the second cycle of loading, and thus the steel plates could bear little tensile stress. In J2G2, however, it was different from J2G1 in that J2G1 did not have a continuous steel plate across its column, The steel plates of J2G2 could bear both tensile and compressive stresses since they were placed continually by being glued on the lateral surfaces of the beam, and its bearing capacity was much larger than that of J2 or J2G1 though it also developed plastic hinges at the beam ends just as J2 and J2G1 did.



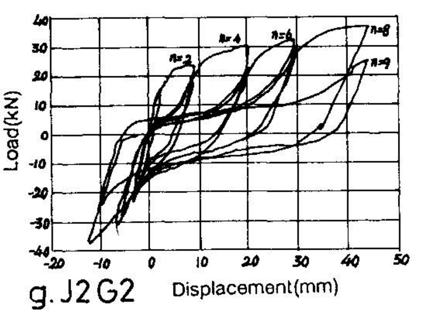


Fig.4. Load-displacement curves

Joints	Δy (mm)	Δu (mm)	μΔ
J1	5	14	2.8
J1G1	7	45	6.4
J1G2	5	38	7.6
J1G3	8	48	6.0
J2	5	45	9.0
J2G1	2.5	18.5	7.4
J2G2	8	48.5	6.0

Table 3. Displacement Coefficients of Ductility

3.2 Bearing capacity under seismic loads

The yield loads and the ultimate loads of different specimens are given in Table 2. The ratios of the loads after being strengthened to those before being strengthened are also presented in Table 2. All the strengthening methods in SET I had greatly improved the shearing strength at the cores, and both the yield and the ultimate loads were increased remarkably compared with those of J1. In SET II, however, only J2G2 had greatly improved the flexural strength of the beam ends while J2G1 almost remained unchanged because of the anchorage failure between the glued steel plates and the column.

3.3 Load-displacement curves

From the recorded data of the loads and their relative displacements at the free ends of the beams, the load-displacement curves of each specimen can be graphed as presented in Fig.4. As $\mu\Delta=\Delta u/\Delta y$ where $\mu\Delta$ is the displacement coefficient of ductility, Δu is the ultimate displacement and Δy is the displacement when the beam-column joints yielded, it is possible to calculate the $\mu\Delta$ values. these values are presented in Table 3. From the curves shown in Fig.4 and the values of $\mu\Delta$ in Table 3, it is very clear that the seismic behavior in SET I had been greatly improved after the joints had been strengthened, because the shearing strength and deformation capability had been increased remarkably and their displacement coefficients of ductility were also much larger than that of J1. In J1G1, since the displacement in the positive direction in the sixth cycle went so far away from the control that much more plastic deformation had developed in this direction than expected, consequently the peaks in this direction of the following cycles of loading were much lower than that of the sixth cycle. It may be supposed that the curves in this positive direction would have been more reasonable if there were not so much displacement in the sixth cycle. In SET II, the curves in J2G2 are much more precipitous than those of J2, and the flexural strength developed was much larger than that of J2, which means that this strengthening method can not only increase the carrying capacity but the flexural rigidity as well. However, it still had a reasonable value for the displacement coefficient of ductility needed for the beam to develop suitable deformation. As to J2G1, it is necessary to improve this kind of strengthening method.

4. CONCLUSIONS

These trial experiments have successfully made comparison between different strengthening methods with externally glued steel to improve both the shearing strength of the cores and the flexural strength at the beam ends. In SET I, all three strengthening methods can greatly improve the seismic behavior of the cores, although J1G2 presented a more reasonable failure mechanism than that of J1G1 or J1G3. However, J1G2 needs more complex construction than those of the latter. When the methods of J1G1 or J1G3 are adopted, suitable adjustment is needed to avoid abrupt failure such as the compressive yield at the column sections or the shearing failure at the cores.

In SET II, the strengthening method of J1G2 can greatly increase the flexural strength of the beam ends, while the method of J2G1 was a failure, and requires further improvement.

5. ACKNOWLEDGEMENT

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