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Hysteresis model for the shear behaviour of R/C structures and its application in the nonlinear response analysis of the 9-storey frame buildings with diaphragms heavily damaged in the 1988 Spitak earthquake

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ABSTRACT: The new hysteresis model describing the shear behaviour of R/C structures is developed on the basis of experimental results obtained from the horizontal load reversal tests of diaphragms (shear walls). The principle and rules for the formation of the hysteresis model are formulated. Cracking, yielding and ultimate points on the envelope curves for the structures of different types are determined. With the help of the suggested hysteresis model nonlinear earthquake response analysis of 9-storey R/C frame buildings with diaphragms was carried out to investigate the reasons of their extensive collapses after the destructive Spitak earthquake in Armenia. Two types of these buildings ("R" type with solid diaphragms and "S" type where diaphragms have door openings) were calculated in longitudinal and transverse directions with consideration of the influence of nonstructural elements, dead loads and dynamic effect on the horizontal stiffness. Calculations were carried out using accelerograms recorded during the earthquake in the towns of Gukasian and Yerevan. Mass destruction of buildings was main caused by undoubtedly huge force, unusually long duration and unfavourable spectrum of the earthquake.

I HYSTERESIS MODEL FOR THE SHEAR BEHAVIOUR OF R/C STRUCTURES

The extensive experimental data obtained by the author on different diaphragms were used to develop a hysteresis model for R/C structures with predominance of shear deformations. The envelope curve can be represented with four linear broken lines (see Figure Ia). Necessary to say that enough experimental data have not been, so far, accumulated for a determination of descending limb of the envelope curve and hence further investigations should be carried out for its clarification. It is assumed that the envelope curve is symmetric about its origin.

In order to explain the principle of the proposed model, schemes are shown in Figure Ib. When the force and displacement exceed the values of FC and DC which represent the coordinates of the cracking point, nonlinear behaviour of construction begins. At this time, a crack may occur in a structure when the displacement is DD_I. When the unloading happens at this moment, the unloading line, as experimental results show, may be taken parallel to the initial stiffness line until the FC level. Then the direction of unloading line changes to the point (-FC, -DC). When the unloading line crosses the horizontal axis (force is equal to zero), the crossing point (0, DR^+) characterizes the residual deformation of the structure. At this moment, the crack has not closed and width of its opening is reduced. The crack will close completely when the structure is loaded in the opposite direction and the loading line reaches the vertical axis (horizontal displacement is equal to zero). Then, with the increase of horizontal force the squeezing of this crack takes place and a new crack occurs in the direction perpendicular to the first. Here the displacement DD_2 is larger than -DC. If the unloading begins at the displacement DD_2, the structure will have a residual deformation DR^- when the horizontal force is equal to zero and after that a new loading cycle begins. The crack which occurred in the previous cycle starts to relax and it causes the reduction of loading stiffness in comparison with the initial stiffness. Therefore loading line strives for the point (FC, $DC+DR^+$) which is considered as a new cracking point. When the loading line crosses the vertical axis, the previous crack begins to open and at the displacement $DC+DR^+$ the increase of the crack width and the development of crack length simultaneously occur. It means that new parts of wall body adjoining with the crack and other undamaged parts are involved in the work under the horizontal force. As a result, the stiffness increases and the loading line changes its

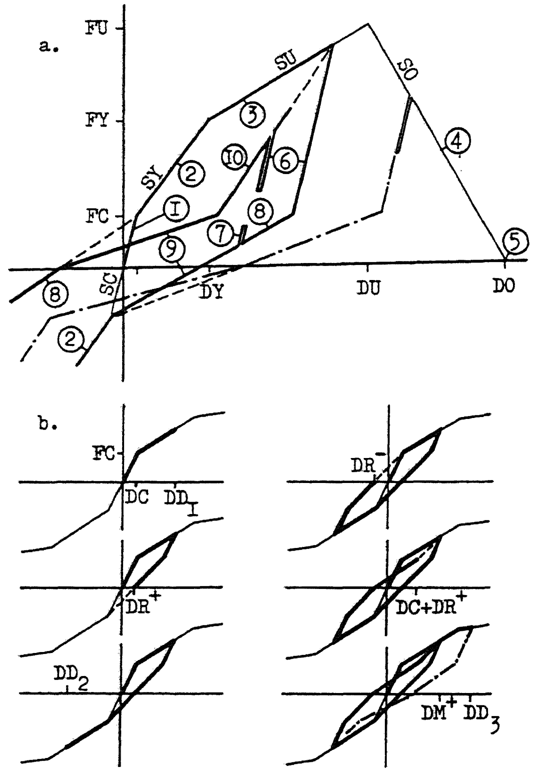


Figure I. Schemes to explain the principle and the rules of the hysteresis model

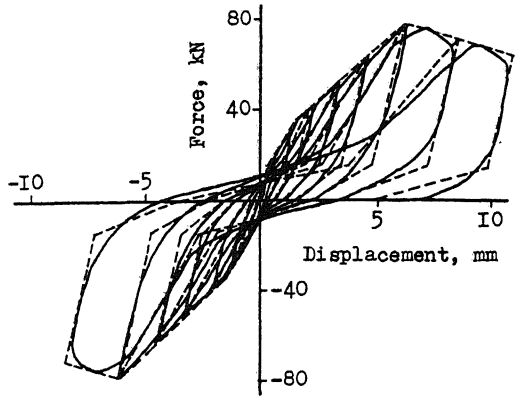


Figure 2. Comparison of the experimental hysteresis loops (———) and the model (------)

direction to the point corresponding to the maximum displacement DM^+ in the previous cycle. Then the loading line goes along the envelope curve up to the displacement DD_3 where the unloading begins and so on.

With the increase of load, new cracks appear which cross each other and cover whole diaphragm body. It is assumed, that the described deformation process takes place up to collapse. Following this principle, ten rules for the hysteresis model formation were defined. In the Figure Ia the numerals in circles denote the numbers of the rules. For comparison, the experimental hysteresis loops and model are combined and shown in Figure 2. From here it obvious that proposed model is close enough to describe the shear behaviour of diaphragm under horizontal loadings.

2 DETERMINATION OF THE ENVELOPE CURVES CHARACTERISTIC POINTS

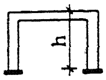
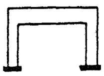
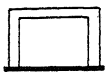
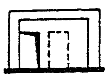
The development of a hysteresis model and rules for its formation is, however, insufficient for earthquake response analysis of multistory buildings. It is also essential to determine the characteristic points on the envelope curves, corresponding to the points after which qualitative changes occur in structures as a result of previous quantitative accumulations of deformations.

Determination of characteristic points also was carried out on the basis of experimental data which were obtained during R/C I:5 scale model structure tests. Geometrical sizes and reinforcement of the specimens correspond to the 9-storey buildings erected in Armenia. In accordance with the design concept, these buildings are categorized as a frame system in the longitudinal direction, and as a braced system in the transverse direction. In other words, the frames have strong beams which support the slabs in the longitudinal direction and diaphragms and weak beams height of which is approximately equal to the thickness of slabs in the transverse direction. There are two types of diaphragms panels - solid and with door openings. Characteristic points were determined not only for diaphragms but also for frames with weak and strong beams. It was revealed from the experimental results that the vertical load has substantial influence on the stiffness, strength and ultimate displacement. Taking account of less influence toward the top of the building, the influence of vertical load on the ultimate displacements was, however, ignored to simplify the subsequent calculations and the ultimate displacements were assumed same as those without vertical loads. From this point of view, the average values of experimental data were derived and the parameters which allow to determine the characteristic points on the envelope curves for different types of structures were calculated (see Table I).

3 EARTHQUAKE RESPONSE ANALYSIS OF 9-STOREY R/C FRAME BUILDINGS WITH DIAPHRAGMS

The calculation model of these buildings may be assumed as a system with absolute rigid slabs. Legitimacy of this assumption was

Table I. Parameters for the determination of characteristic points on envelope curves

Type of structure	DC	DY	DU	$\frac{FU}{FC}$	$\frac{FU}{FY}$	$\frac{FY}{FC}$	$\frac{DU}{DC}$	$\frac{DU}{DY}$	$\frac{DY}{DC}$	$\frac{SC}{SY}$	$\frac{SC}{SU}$	$\frac{SY}{SU}$
	$\frac{h}{I260}$	$\frac{h}{I60}$	$\frac{h}{40}$	7.60	I.69	4.50	3I.25	3.99	7.83	I.95	7.55	3.87
	$\frac{h}{I480}$	$\frac{h}{200}$	$\frac{h}{50}$	7.I0	I.69	4.20	30.00	4.00	7.50	2.03	7.76	3.82
	$\frac{h}{2000}$	$\frac{h}{300}$	$\frac{h}{80}$	6.80	I.70	4.00	25.00	3.75	6.67	I.89	6.55	3.47
	$\frac{h}{3I50}$	$\frac{h}{550}$	$\frac{h}{I50}$	6.70	I.8I	3.70	20.70	3.65	5.69	I.67	4.83	2.89

proved by comparisons of calculated results and experimental investigations. In particular, it was stated that slabs work together with beams up to the collapse stage and that it is necessary to take account of the slab contribution from each side of beam within the width equal to the half of a span length for a determination of the beam stiffness. In this case, it is not difficult to show that calculated values of the building's vibration period will be very close to those which were obtained on the assumption of absolute rigid slabs. It should be noted, however, that the ignorance of slabs may result in significant errors in a determination of vibration periods. Thus, for earthquake response analysis of 9-storey R/C buildings with diaphragms, the calculation model was assumed as a system with masses concentrated in the levels of slabs which move only in horizontal planes. Usually in design practice only structural elements are considered in calculation and the influence of nonstructural elements, dead loads and dynamic effect of loading on total storey horizontal stiffness are not taken into account. However, experiments show that mentioned factors play important role in the forming of seismic forces. Nevertheless until now they haven't been taken into account what lead to not correct determination of vibration periods and consequently of inertia forces on buildings.

In this paper mentioned factors were considered in the following. In accordance with experimental results for R/C frame buildings nonstructural elements bring to magnification of the total storey initial horizontal stiffness in I.6 - I.8 times. For frame buildings with diaphragms magnification coefficient is equal to I.I5-I.35 dependimg on the type of diaphragms. Dead loads bring to magnification of the columns stiffness in I.I-I.3 times and of the diaphragms stiffness in I.I-I.6 times depending on their place in the building plan and the number of floors. Influence of dynamic effect also was taken into account on the basis of experimental data using coefficient I.2 for increasing of the horizontal initial stiffness of structural elements.

Earthquake response analysis was carried out with the program which was developed by T. Inoue at the laboratory of Professor T.Okada and Y.Nakano of the Institute of Industrial Science, University of Tokyo. In accordance with Table I and initial stiffness input data for displacements and forces corresponding to the cracking, yielding and ultimate points for columns, diaphragms and nonstructural elements were prepared. Damping coefficient was taken 4% of the critical. For nonstructural elements it was assumed that the stiffness changing displacements were the same as those of diaphragms but that the collapse immediately after yielding because of their poor ductility properties.

In this analysis, two types of 9-storey buildings were analyzed; with solid diaphragms and with diaphragms including door openings (see Figure 3). Calculations were carried out in the transverse and longitudinal directions. The proposed hysteresis model was used for diaphragms and Degrading Tri-linear hysteresis model for columns.

Because of extremely serious housing situation in Armenia now the realization of seismic protection of existing buildings without eviction of lodgers is highly desirable. For this aim it was proposed to construct on the existing buildings the flexible upper floor as a vibration damper. Therefore here corresponding versions of buildings with vibration damper were also calculated.

In the epicentral zone of Spitak earthquake instrumental records of ground motion were not obtained unfortunately. But there are two records of ground acceleration obtained in Gukasian by accelerograph "CCP3" belonging to the Institute of Geophysics and Engineering Seismology, Armenian Academy of Sciences and in Yerevan by accelerometer

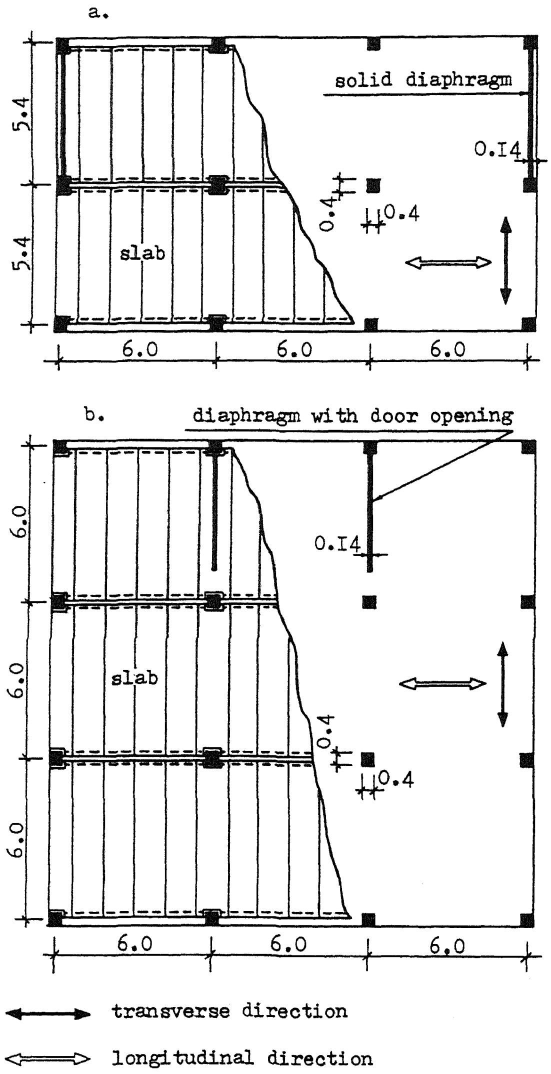


Figure 3. Plan of the 9-storey frame building
a - with solid diaphragms ("R" type);
b - with diaphragms including door openings ("S" type)

"ОСП-2М" belonging to the Scientific Research Institute of Civil Engineering and Architecture, Armenian State Committee of Construction (see Figure 4). Both records were used herein for earthquake response analysis of the 9-storey buildings. The intensity of Spitak earthquake during the main shock was IO on MSK scale. Then aftershocks with intensity 8, 9 and 9 followed. Therefore calculations were carried out herein using not the original but the magnified accelerograms with a peak acceleration of 200, 400 and 800 gal which corresponds to intensity 8, 9 and IO, respectively.

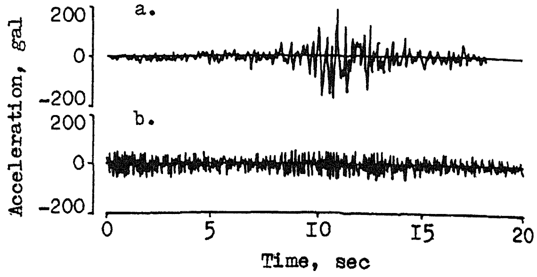


Figure 4. Spitak earthquake accelerogram recorded at Gukasian (a) and Yerevan (b)

4 RESULTS AND DISCUSSION

Calculation results with respect to the vibration periods are to testify that assumption about absolute rigid slabs brings to 89% reduction of periods in average. Combined consideration of nonstructural elements, dead loads and dynamic effect brings to reduction of periods on 40% in longitudinal direction and 30% in transverse. According to the experimental data first mode vibration period of that buildings is equal to 0.65 sec in longitudinal and 0.49 sec in transverse directions, but average calculated periods are equal to 0,7I sec and 0,5I sec, respectively. Consequently, difference between results obtained here and real values is very small. Thus, at the formation of the calculation model for frame buildings with predominance of shear deformations it is advisable to accept slabs absolute rigid and take into consideration influences of nonstructural elements, dead loads and dynamic effect at the determination of the buildings total horizontal stiffness.

Under the Gukasian accelerogram action at the peak accelerations 200, 400 and 800 gal for both buildings the response shear forces exceeded design forces 2.I, 3.4 and 4.6 times in average, respectively. Under the action of Yerevan accelerogram exceeding was I.4, 2.5 and 3.4 times, respectively. It has been shown, as well as data about deformation stages, that the action of the Gukasian accelerogram was more unfavorable. The reason becomes obvious if to compare the Fourier spectrums of both accelerograms (see Figure 5). It is easy to notice that predominant periods of the ground motion at Gukasian are rather close to the calculated buildings first mode vibration periods. Despite the significant exceeding of the design forces collapse was obtained mainly of the nonstructural elements. In structural elements took place the intensive development of plastic deformations (yielding stage). Maximum ductility factors were for columns 3.28, for solid diaphragms 2.78 and for diaphragms with door openings 5.68 under seismic action corresponding to intensity IO. For intensity 9 - I.37, I.18 and 2.54, respectively. From Table I the ratios DU/DY are equal

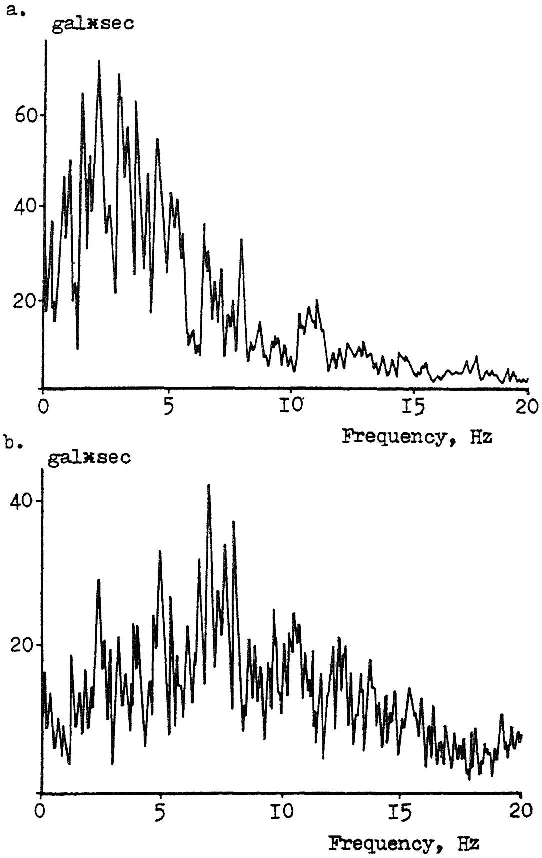


Figure 5. Fourier spectrum of Spitak earthquake observed at Gukasian (a) and Yerevan (b)

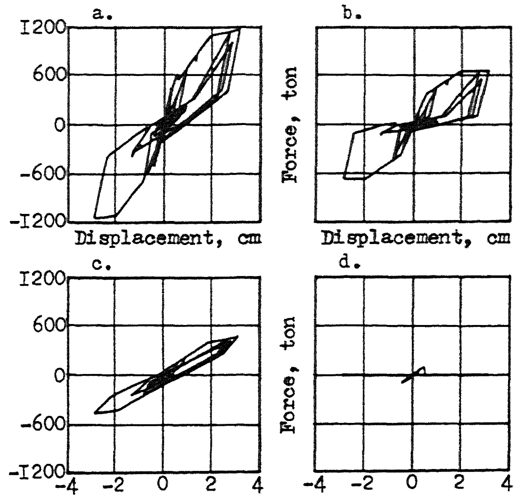
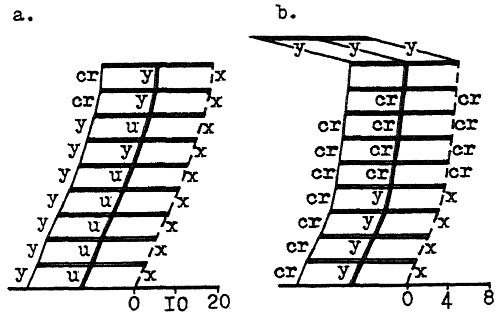


Figure 6. Restoring force - displacement relationship for the first floor of the "S" type building under the Gukasian accelerogram action a - total; b - for diaphragms; c - for columns; d - for nonstructural elements



cr, y, u, x - cracking, yielding, ultimate and collapse stage, respectively
—— column ▬▬ diaphragm
---- nonstructural element

Figure 7. Deformation state of the "S" type building under the Gukasian accelerogram action
a - without flexible top storey;
b - with flexible top storey (vibration damper)

to 4.00, 3.75 and 3.65, respectively. Thus, at intensity I0 the diaphragms with door openings were in ultimate stage (see Figures 6 and 7a). But here is very important to bear in mind the following. In the calculations the effect of dead loads on the stories shear forces and overturning moment through the lateral displacements of the buildings (commonly called P-Δ effect) was not taken into account. Then, the bad quality of construction was not considered. At the same time, as it has been mentioned above, the main shock was followed by another three shocks with very high intensity. But calculations here were done only under the main shock action. If to consider the influence of all these factors on the behaviour of the buildings and believing that the intensity of the main shock in Gumry (Leninakan) was I0 it is possible to certify that destructions of buildings, especially of those where diaphragms had door openings, were unavoidable.

Under the seismic action corresponding to intensity 8 structural elements were in cracking stage. That result also reflects the real behaviour of the buildings during the Spitak earthquake very well. Enough to remember that in Kirovakan where intensity was 8 no one building collapsed. This prove again that undoubtedly huge force and unusually long duration of the earthquake were the main causes that brought buildings to massive destruction.

In the transverse direction of the building with rectangular plan solid diaphragms are perceive about 70% of horizontal force, columns - 20% and nonstructural elements - I0%. In the transverse direction of the buildings with square plan diaphragms with door openings are perceive 60% of horizontal force, columns - 30% and nonstructural elements - I0%. In longitudinal directions of both buildings nonstructural elements are perceive 30% of horizontal force. It means that nonstructural elements have noticeable influence on the buildings behaviour. In the further investigations and design more attention should be pay to them.

One of the possible ways for further exploitation of existing buildings under conditions of increasing the seismicity of the territory of Armenia up to intensity 9 is the using of vibration damper in the form of flexible top story. In accordance with calculation results one-mass damper brings to reduction of the shear forces in the transverse direction of the "R" type building in I.8 times and in longitudinal direction - I.5 times in average. For the "S" type building - in I.6 and I.3 times, respectively. Reduction of the horizontal displacement on the level of nine storey for both buildings is 50% in average. Reduction of forces on the upper stories is more than on lower stories. It is necessary to notice that because of the flexible top storey significant decreasing of deformation stages level in structural elements also took place (see Figure 7b). But damper's structures come to yielding stage. It is quite permissible because the flexible top storey is not a dwelling one.

5 CONCLUSIONS

On the basis of experimental results the hysteresis model was proposed to represent the shear behaviour of R/C structures. Characteristic points on the envelope curves for different types of structures were determined. With the help of proposed hysteresis model earthquake response analysis was carried out for two types of 9-storey buildings which had extensive damages due to I988 Spitak earthquake in Armenia and the following main conclusions were obtained.

I. Mass destruction of buildings was main caused by undoubtedly huge force, unusually long duration and unfavourable spectrum of the earthquake. At the ground motion acceleration of 400 gal and 800 gal, corresponding to the intensities 9 and I0 by the MSK scale, the load affecting the buildings exceeded the design force 3.4 and 4.6 times, respectively.

2. At this loading level, even without considering the P-Δ effect and ignoring the bad quality of construction, structural elements were mainly at the yielding stage while diaphragms with door openings were found at the ultimate stage. Nonstructural elements collapsed. Taking into account that the deformation stages, mentioned above, manifested themselves immediately after the first shock, it may be certified that heavy damages and destruction of the buildings could not be avoided after the three strong shocks that followed.

3. At the acceleration of 200 gal, corresponding to the design intensity 8, the response shear forces exceeded the design forces more than two times. At this loading level structural and nonstructural elements were mainly at the cracking stage.

It indicates the necessity of revising the existing seismic standard in order to clarity the method of the seismic force determination. It is also necessary to revise construction requirements, aiming to provide a greater ductility of structures.

4. A comprehensive on-site inspection of the existing buildings must be carried out in order to find optimum ways of their protection against possible earthquakes in the future. Simple decrease of a number of storeys in the existing buildings, especially those erected on hard soils, may lead to negative consequences.

5. Forming a calculation model for frame buildings with predominance of shear deformation, slabs should be assumed absolutely rigid. When the building horizontal stifferess is determined it is also advisible to take into account the influence of nonstructural elements, dead loads and dynamic effect. Combined consideration of these factors results in the vibration period reduction by 30 to 40%. Nonstructural elements noticeably affect the building behaviour.

6. Further exploitation of the existing buildings without evacuating the lodgers may be possibly provided by application of the flexible top storey as a vibration damper. One mass damper reduces shear forces I.7 and I.4 times in longitudinal and transverse directions, respectively. Reduction of the horizontal displacement on the level of the nine floor is I.5 times.

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