BEHAVIOUR OF STRUCTURES DURING EARTHQUAKES BEYOND THE LIMITS OF THE ELASTIC STAGE.

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SUMMARY

The results of the experimental study of the strength and stability of the welded steel frame cages with joints of different types under static and dynamic loads in life and of the study of the mechanical oscillators of soft and brittle steel and of the pressed sand concrete under dynamic loads of the stationary nature and of the "single" bounds type depending on their stressed deformed state are described.

Introduction

As it is known, seismic loads conditioned by the corresponding standards, characterize the intensity of the earthquakes in the average degree and to a certain extent, conventionally. But the extreme instability of the seismic accelerations can be easily displayed by any accelerogram: naturally, for this reason arises the question: which value of the acceleration should determine the intensity of the earthquake,i.c. its force in balls. Reliance on the maximal values of the accelerationscan involve orientation on the excessive reserves of the bearing capacity of structures while decrease of the seismic load down to the level of the average intensity of the accelerations in same moments will cause the overloads 2-3 times exceeding the design assumptions. That is why the investigation into the behaviour of structures beyond the limits of the stage of the elastic performance acquires great practical importance.

Let's dwell on the behaviour of the metallic structures. Here it will be appropriate to pay attention to the fact that the use of the high-plastic material in structures does not yet exclude the possibility of their brittle failure as all the junctions carrying the maximal stress in the metallic st structures, are made of weld for which reason the welded joints and the abutting metal become frail. As the structures are extremely frail just here, it is lekely that damage will occur in them and none of the plastic deformations will develop in the process.

To make the plastic deformations in the frame cage possible it is necessary to remove the dangerous sections of the structure from the zone adjoining the weld. It can be done by different ways (Figs.1b and 1c). In ding so the spots of transition to the reinforced section must be selected so as the

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stresses in the welded joint of the cross-bar and the strut would not exceed the rated resistances while the plastic hinge appears in the plastic zone. With the appearance of the plastic hinge an increase in efforts in the space between the hinge and the joint of the cross-bar and the strut must stop and in this way the plastic hinge will be the kind of protector of the welded joint from the overstress.

The second factor which can become an obstacle to the use of the plastic dimensions of the structure, lies in the danger of loss in the stability of the bearing structures which can come before the stage when the stresses will reach the point of yield G_{yed} for metallic structures of the thin-walled members the danger is aggravated by the possibility of the loss of the local stability as the latter can happen not only in the compressed but also in the bending members.

As the struts are made of the thin-walled members and exposed to the permanent compression, the serious danger to them and to the structure as a whole is threatened by both the loss of the stability of the member on the whole and the local loss of the stability. That is why it is dangerous to have the plastic deformations in the struts. The cross-bars are mainly subjected to bending and the loss of the local stability is less dangerous for them and because of this the plastic deformations in the cross-bars are permissible. But during the vibrations of the building in the moment of the appearance of the plastic deformations in the cross-bars an increase in the efforts in the cross-bars will cease and due to this the seismic load will be overdistributed on the struts. Lest it should cause the overload of the struts the plastic deformations in the cross-bars must be limited.

To valize the experimental check of the formulated considerations the tests of the frame joints of the framed building have been conducted. To this effect the load P causing the bending in the cross-bars, was tied to their ends. The tests have been berformed under the static and dynamic loads. The dynamic loads were both simple and alternating.

The diagrams P-y constructed during the static tests,by their shape proved to be close to the ideal diagram of the elasto-plastic deformation - Prandtle's diagram.

In the samples of A type the zones of the plastic deformations of the cross-bars were formed just behind the plates connecting the cross-bars to the struts. As it was expected in the samples of B and C type the zones of the plastic deformations appeared in the spots of transition from the basic section to the reinforced one. In the process there occured not any marked damage in the points of the joints of the cross-bars and the struts.

None of the samples was brought to the complete failure under the static loads - the attempts to do this were unsuccessful though the potentialities of the experimental device could enable one to bring up deflections approximately to 15 Yyield (Yyield=ultimate elastic deformation of the cross-bar).

The analysis of the results of the tests and the examination of the tested samples showed that development of the plastic deformations in the cross-bars was accompanied by the loss of the local stability in the compressed fibers of the cross-bars which took place during the deformation in the plastic range. At first the loss of the local stability was observed in the compressed flange of the cross-bar but it did not have an appreciable influence on its bearing capacity. Afterwards the bearing capacity of the cross-bar diminished when the web lost the local stability.

Here it should be noted that in the samples of C type having comparatively high web in the supporting piece of the cross-bar the loss of the bearing capacity was already observed when the value of the deflection of the cross-bar was only 4-5 Yyield and thisn occurred because of the loss of the local stability of the web. As for the samples of A and B type in which the webs of the cross-bars were small it is to be said that owing to this fact their local stability was higher and there was not a distanct reduction in their bearing capacity even when the deflections equalled to 15 Yyield.

As a result of the dynamic tests, the failures in the samples of A type occurred in the area of the welded joints despite of the fact that they had widened connecting plates which provided some trop in the stresses. In the process much lesser number of the loading cycles was used to destroy the samples than in the case of the samples of B and C type. At it should be expected the failures of the cross-bars in the samples of B and C type took place in the plastic zones removed from the welded joints and the area of the welded joints remained quite undamaged.

More reliable performance of the cross-bars of B type as compared to those of A type is clearly displayed by the graph in Fig.2 where Pd/Pst - lg n relation ships are shown. Both the correlational straight lines are almost parallel but the straight line relating to the samples of B type runs considerably higher. In the cases when the intensity of the load applied to lied to the cross-bars, reached the maximum vamue P yield, the total en total energy capacity [w] more than 200 times exceeded the value of the maximal potential energy υ. With the decrease in the applied load P_d the energy capacity would become still higher and even at the relation Pd/P = 0.7-0.9 the value of [w] up to 600-800 times exceeded So, [w] = 200 v was the lower boundary of the energy capacity which is possessed by the cross-bars of the given type if the zone of the plastic deformations is sufficiently far reloved from the welded joints.

The energy capacity of the samples of A type proved to be much less than that of the samples of B and C type. For instance when the intensity of the dynamic load was close to P_d

Pyield, in one case the sample was destroyed after 6 loading cycles at Pd=0.92 and in another its failure happened after 10 loading cycles. The consumed energy was there with 32 σ and 72 σ respectively. But in the only case when Pd=0.84 P_T Pyield the failure occured after 45 cycles and the volume of the consumed energy anoubted to $[w]$ =240 σ.

So, the tests have shown that the structures in which the plastic zones, due to the enlarged support sections are sufficiently for removed from the welded joints, can with stand the action of the repeated dynamic load for a long time.

It was concluded from the obtained experimental data that the estimation of the bearing capacity of the bendable members should be made from 2 points of view. Firstly, the structure must meet the requirements of strength under the repeated seismic vibrations in the range of the stressedstage going over the limits of the elastic performance of the material and, secondly, the maximum deformations of the web of the beam must not exceed the values ensuring the stability of the web.

In the first case the condition of the structure's stability is determined on the basis of the energetic considerations [I] by the inequality:

$$\frac{M_z}{2\,c} - \frac{M_{yield}}{2\,e} \leqslant \frac{[w]}{n} \qquad \mathrm{I}$$

$$\text{or} \quad M_z \leqslant M_{yield} \sqrt{1 + \frac{2\,c\,[w]}{n\,M_{yield}^2}} \qquad 2$$

where M_z and M_{yield} are, respectively, the conventional bearing capacity of the section under the seismic action and the bearing capacity corresponding to the ultimate bending moment during the formation of the plastic hinge.

c = rigidity within the limits of the beam's elastic performance,

n = expected number of the loadings exceeding the elastic. limit of the member.

It became possible to provide the fulfilment of the second condition with the help of the same expression (2) but only through the use of not full energy capacity of the section $[w]$, but of its certain limited amount w_{orp}, determined by the maximally admissible plastic deformation of the web securing its stability. Really, as on account of the moments created by the static working load, the section of the cross-bar will be unsymmetrically loaded even under the symmetrical dynamic loads and the stresses, corresponding to the yield point of the material, will always initially reach maximum in the upper fibres of the section and under the repeated loads the growth of the plastic deformations will be one-sided. Hence, the limited value of the energy capacity will correspond to the maximally admissible plastic deformation securing the web's stability.

Without prosenting here rather complicated and expanded formulas determining the maximally admissible plastic deformation of the section, we shall just note that the value of w_{orp} is much less than the value of [w] and is only 25-30%.

In addition to the above location tests the model tests have been conducted whose aim was to investigate the influence of the plastic properties of the materials and of the stressed-deformed state of structures on their response to the seismic loads.

The mechanical oscillators have been under test (Fig.3) - these oscillators were presented by the simplified models of structures with one degree of freedom and were made of the soft (St-3) and brittle steel and of the pressed sand concrete of "300" brand on the rigid foundation as well.

The tests were performed on the centrifugal test device AzIS-2 [2] . The parameters of the device permit to generate in the system under test practically any stressed - deformed state to failure as because of the normal (in the given case compression) static loads so of the bending inertia loads of the stationary sinusoidal and impulse ("bounds") character.

The method of the tests of this set is the following. The mechanical oscillators are fixed on the bottom platform of the test chamber (Fig.6) and in the process of its rotation with a speed, corresponding to the designated stressed state of the system caused by the compression loads, are subjected to the vibration or "bounds" in the regime conforming to the prearranged stressed-deformed state generated by the bending loads. In the process the vibrations of the test chamber (1) and of the yoke of the mechanical oscillators (2) were fixed with the help of the special measuring heads (3,4), stable to the extremely heavy overloads.

The tests have revealed that both in the conditions of the gravitational field and of the centrifugal field, i.e. irrespective of the value of the normal loads within the limits of the elastic performance the diagram P-Y for the soft steel is close to Prandtle's diagram and the one for the brittle steel and the pressed sand concrete - to that of the elastic deformation.

Other things being equal, the damping coefficient (E) of free oscillations significantly depends on the stressed-deformed state of the system created by the bending and compression loads.But there exists no rectilinear relation ship between the stressed-deformed state of the systems and the change in the damping coefficient and the sharp increase in the damping is mainly observed during the system's transition from elastic stage to the plastic or the elasto-plastic one, reaching its maximum at G=0.8-0.9R, where R= ultimate strength. The cause of this is likely to be found in the structural change at the molecular level.

The data obtained in the test process from the results of the telemetric measurement and the observations in the process of the deformation and destruction of the systems' models have demonstrated that:

- the ruptures of all the samples of the soft and brittle steel and of the pressed sand concrete as well took place along the sections adjacent to the supporting part of the beams;
- the development of the plastic deformations of the oscillators of the soft steel was accompanied by the loss of the local stability of the compressed fibres of the beam, occurring during the deformation in the plastic range;

- all things being equal, including the stressed-deformed states generated by the compression loads, the destruction of the samples of the soft steel requires several (up to 15-20) times, more the loading cycles than the destruction of the samples of the brittle steel and the pressed sand concrete and the energy capacity of the former samples is respectively much higher than that of the latter ones.

So, the location and model tests, correlating well between them, point to the presence of the considerable reserves of strength under the loads of the seismic type in the structures of the materials having a distinct elastic "site" and to the practical possibility of using these reserves.

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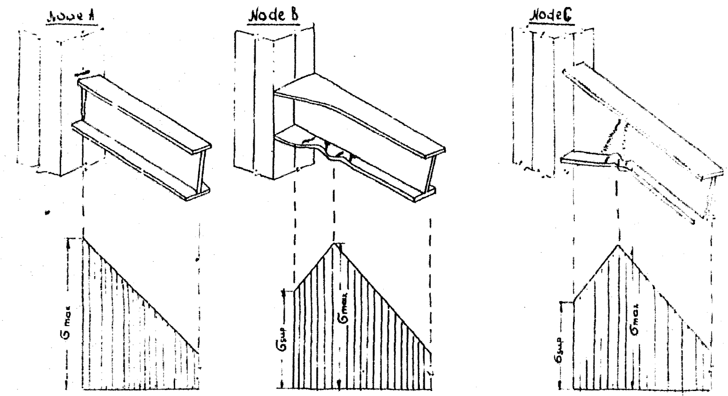
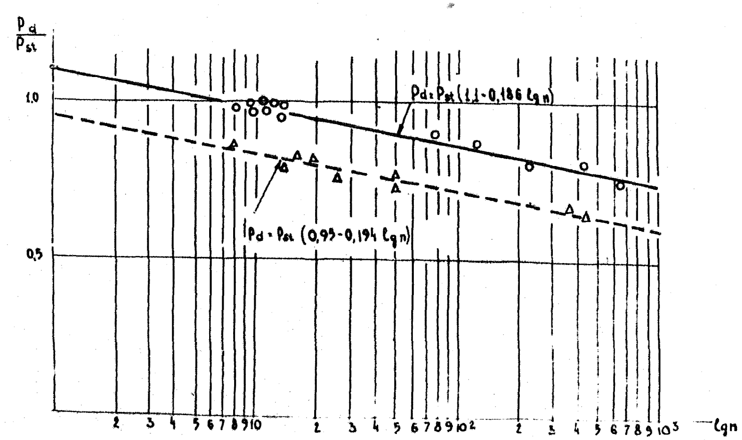


Fig.1 - Scheme of the Structural Solutions of the frame joint and the Curves of the Stresses in the Gross-Gar



Fig;2 - The diagram of the Dependence of P_d/P_{st} upon Lgn for the Gross-Gars of the Types A(--) and B(-).

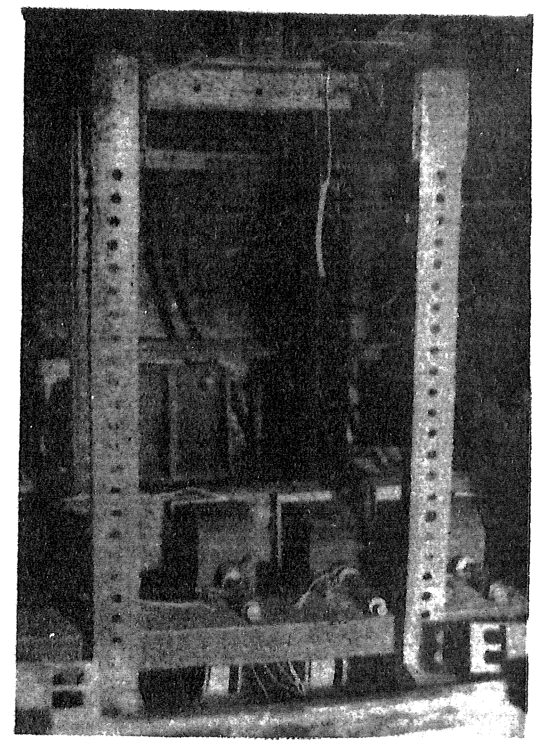


Fig.3 - General View of the test Chamber with the Simplified models of Structures