ON REDUCING SEISMIC LOAD, SELECTING OPTIMUM RIGIDITY OF STRUCTURES AND SPECIFYING SURPLUS SEISMIC RESISTIVITY

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SUMMARY

Considerations are suggested for decreasing seismic loading at earthquakes by using light-weight structural materials with a higher vibration damping, experimental results are presented on the vibration of light-concrete structures having the said properties.

Presentation is made of the problem on selecting optimum rigidity of a frame building with a given, determinated law of the foundation displacement. The optimum criterion is taken to be the weight or the cost of the bearing structural units. The study resulted in the optimum characteristics being alotted to the buildings with vertically diminishing rigidity.

Experimentation on the behaviour of the ferro-concrete frame-type structures under impulsive loading has resulted in the determination of their surplus seismic resistance.

1. Reducing seismic stress

One of the basic principles of securing seismic resistance of structures is minimizing their exposure to seismic stress through weight reduction and by using materials with higher vibration-dissipation characteristics.

An effective weight reduction of structures is attained by the use of soft and reinforced concrete producing weight reduction of 25-40 per cent depending on concrete density. Following these guidelines the Armenian SSR carries out a major part of its civil, hydrotechnical and transport construction using light concrete and reinforced concrete with natural porous aggregates.

Quantitative data on damping vibration in light ferro-concrete have been obtained by experimentation on the decrement of oscillations for the deflectable units of light as compared to heavy ferro-concrete having differing amounts of longitudi-

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nal reinforcements at dynamic and static loading (with no longitudinal compression force, with centrally acting longitudinal force and eccentric compression at varying eccentricities.

The experimental specimens were ferro-concrete columns 180 cm. high, 14 x 20 cm. at cross-sections and with longitudinal reinforcement: 0.66; 0.95; 1.94; 2.53 per cent.

Tests were conducted on a seismic platform producing the needed extensive accelerations in a wide range of frequencies. The platform was supplied with special devices for reliable fixation of specimens and to enable both central and eccentric application of the longitudinal compressing force.

Each column held a wire strain gauge having a base of 20 mm. on all reinforcement rods and having a base of 100 mm. on the concrete at four vertical levels on both sides of the column. Beside strain gauges for measuring deformations there were also gauges mounted for monitoring displacements of the platform and the top end of the column.

The specimens were first tested for transversal bending (dynamic and static) with no compressing force and then with centrally applied compressing force. Further tests included transversal bending with eccentric compression, the eccentricities being h/6 and h/2, where h is the section height of the columns. The tests have been conducted with the same cycles and the same strains as in the case of the transversal bending with no longitudinal force and with central compression.

To establish the bearing capacity of the specimens one of them was forced to destruction with constrained oscillations in the resonance mode of operation, the platform acceleration being 3.41 g. Subsequently taking the maximum acceleration of the platform to be 60 per cent of the destructive, determinations were made of the platform operational accelerations for the whole series of testing. The major testing was then started for the constrained oscillations with differing platform accelerations (0.94; 1.35; 1.77 and 2.18 g).

Processing the emerging recordings resulted in the presentation of the resonance curves, the decrement of oscillations for all specimens tested being determined by the width of the curves.

Next, testing was conducted for free transversal oscillations caused by an instantaneous disengagement of the specimen from the horizontally applied force. The maximum amplitude of the induced free oscillations agreed with the resonance amplitude of the constrained oscillations. The resulting recordings yielded the decrement of oscillations using trivial methods.

Next series of experiments included testing the transversal bending of the specimen with the repetitive alternating static loading applied in steps up to the values originating initial specimen saggings during free oscillations.

The resulting data enabled plotting of hysteresis loops for the full load-unload cycle to further yield the decrements of oscillations.

The analysis of the resulting data has produced the following regularities:

The decrement of oscillations for ferro-concrete columns at all testing stages and at different levels of loading has come to be greater for light concretes than for the heavy ones. This difference is greatly increasing in the presence of the longitudinal compressing force, particularly so in the cases of its eccentric application as well as with an increasing percentage of reinforcement, being contained within 23 to 63 per cent.

During the transversal oscillations and in the case of the centrally applied longitudinal compression force the decrement of oscillations has come to be smaller as compared with the transversal oscillations with no longitudinal force applied.

This decrement is particularly tangible at greater percentages of reinforcement. Thus, at $\mu = 1.94$ for light concretes this decrement attains 16 per cent and 40 per cent for the heavy concretes.

For the transversal oscillations in the case of an eccentric application of the compressing force the decrement of oscillations is smaller compared to the centrally applied longitudinal force by 11-28 per cent (the greater percentage of decrement corresponds to the heavy concrete).

At all events during transversal oscillations the decrement of oscillations is dropping with an increased percentage of reinforcement (for the same stresses):

- with no longitudinal force - 0.49 to 0.25 for the light concretes and 0.39 to 0.19 for the heavy concretes;

- with longitudinally applied compressing force - 0.47 to 0.22 for the light concretes and 0.27 to 0.16 for the heavy concretes;

- with longitudinally applied compressing force having insignificant eccentricity - 0.46 to 0.21 for the light concretes and 0.35 to 0.13 for the heavy concretes;

- with longitudinally applied compressing force having a significant eccentricity - 0.44 to 0.20 for the light concretes and 0.29 to 0.12 for the heavy concretes.

This conclusion is important for reducing metal consumption.

During transversal oscillations with any reinforcement percentage and in all of the cases considered (with no longitudinal force, with centrally or eccentrically applied longitudinal force) the decrement is increasing with an increased acceleration of vibration. In the case of no longitudinal force this increase constitues. 10-20 per cent for the light concretes and 15-30 per cent for the heavy ones. With the centrally applied longitudinal force this increase constitutes 12-24 per cent for the light concretes and 12-32 per cent for the heavy ones. With eccentrically applied longitudinal force this increment constitutes 11-25 per cent both for the light and heavy concretes.

Thus, application of light concretes results in reducing the seismic loading both through reducing structural weight and by improving vibration damping.

2. Selecting an optimum rigidity of frame buildings

The problem considered is the one on the optimum selection of rigidity for a frame building with a known determinated law of foundation displacement. For a uniform material the criterion of optimum characteristics is taken to be the weight (volume) and for a heterogeneous one the criterion is the cost of the bearing structural elements. In searching for the optimum dimensions of units the geometrical layout of the frame is assumed to have been pre-specified. All units meet the requirements of strength and rigidity.

In some cases, particularly with harmonically oscillating foundations, the problem on selecting the optimum rigidity of a regular frame having the minimum weight is solved in a closed form. The dynamic design scheme is assumed in the form of a system having a finite number of the degrees of freedom. The motion equations are composed using the method of deformations. Use is made of the mathematical apparatus for solving variational problems with no limitations to the values of parameters being variated upon. Introduction of Lagrange factors and composing equations matching the minimum of the specified functional originates a system of equations determining the parameters characterizing the state of deformation and the cross-sectional dimensions of units. In case of a rectangular section and at specified sectional dimensions the optimum dimensions are determined in the manifest form.

The minimum-volume frame system performing harmonic oscillations poses as an equiresistant structure. The maximum normal stresses within each unit are practically equal. There is also equality between the linear and angular deformations of rigid joints at all storeys except the first. The first-floor rigid-joint deformations are somewhat different from the upper ones because of the different boundary conditions of the first-floor units. With the floors having absolute rigidity, the optimum rigidity is characteristic of the frame in which all the floors have equal relative deformations. Solution of an actual example has shown the volume of the optimum double-span ten-story frame to constitute a 0.86-th part of the volume of the same frame having, however, constant rigidity in the vertical dimension.

Let us consider an optimum selection of rigidity of the frame buildings having a shearing pattern of deformation for the cases of non-stationary influences during impulsive foundation displacements and of the displacements following the real-earthquake accelerograms. An optimum resistance to the instantaneous influence (seismic shock) has been offered by the buildings having a vertically variable rigidity expressed by the following equations:

$$a_k = a_1 \left[\alpha + \frac{n-k}{n-1} (1-\alpha) \right]$$

$$\alpha = -(n-2)\,0{,}08 + 0{,}96; \quad n = 2 \div 11$$

where: a_k, a_1 - story rigidities of the k-th and I-st floors,
n - number of storeys.

Consider the optimum problem in the case of using accelerograms of the real earthquakes. The criterion of optimum performance for a reinforced-concrete building is taken to be the cost of the bearing structures. The variable parameters were the geometrical dimensions of the first-storey columns: cross-sectional dimensions and the storey height.

With two real accelerograms a ten-storey reinforced-concrete building had optimum indications in the case of the rigidities of the first and the upper storeys being in close proximity ($a_1/a \approx 0.6 \div 0.7$). In this case the first-storey height in the optimum version was equal to the heights of the upper storeys. The saved cost as compared to the constant-rigidity frame was 4÷8 per cent.

The optimum rigidity can be determined not only by variating the geometrical dimensions but also by an adequate selection of structural material.

It has been shown that for the buildings with predominant bending and shearing deformation the criterions of optimum material for bearing structures are the values

$$\frac{[\sigma]}{(E\gamma)^{1/2}} \quad \text{or} \quad \frac{[\tau]\,K^{1/2}}{(G\gamma)^{1/2}}$$

where $[\sigma]$, $[\tau]$ - respectively admissible normal or tangent stress;
E - elastic modulus;
G - rigidity modulus;
γ - volume weight of the material,
K - a coefficient characterizing an uneven distribution of the tangent stresses (K =0.83 for a rectangle).

3. Specifying surplus seismic resistance

During an earthquake the columns of a ferro-concrete frame are being suddenly displaced with the building foundation thus subjecting themselves to the action of inertial impulse-type forces at the floor levels. The forces (reactions) originating in them attain the maximum values when the duration (τ) of these forces being active through a sudden desplacement of the building foundation becomes commensurable with the half-period of its free oscillation.

At the elastic stage of structural operation $P = S\omega\varepsilon$ where

$$S = P_g \cdot \varepsilon\,; \quad \omega = \frac{2\pi}{T}\,; \quad \varepsilon = \frac{\sin\frac{\omega\tau}{2}}{\frac{\omega\tau}{2}}$$

at $\tau = \frac{T}{2}$; $P = 2P_g$

In this connection there exists a linear dependence between P and the dynamic sagging (y_g), which for the impulsive action is defined by the expressions:

- with unspecified energy absorption

$$y_g = \omega\delta_{11} P \int_0^t \sin\omega(t-u)\,du = y_c(1-\cos\omega t)$$

at $t = \tau = \frac{T}{2}$; $y_g = 2y_c$

- with specified energy absorption

$$y_g = y_c\left[1 - e^{-\alpha t}\left(\frac{\alpha}{\omega}\sin\omega t + \cos\omega t\right)\right]$$

where $\alpha = \frac{\psi}{2T}$ - damping coefficient at $t = \tau = \frac{T}{2}$

$$y_g = y_c\left(1 + e^{-\frac{\alpha\pi}{\omega}}\right) = y_c\left(1 + e^{-\frac{\psi}{4}}\right) = y_c K_g$$

therefore, at the elastic stage of the column operation

$$P = P_g\left(1 + e^{-\frac{\psi}{4}}\right)$$

As a result of the author's examination of ferro-concrete fragments and natural-sized frame columns as subjected to the multiple alternating as well as to maximum impulsive (at $\tau = \frac{T}{2}$) horizontal action forcing them to destruction, it has been established that:

- When effecting the ferro-concrete frame in compliance with the requirements of the building standards and regulations as to concrte strength, technology and selection of reinforcement (mainly lateral reinforcement) the development of elastic and plastic deformations causes reduction of the dynamic coefficient , this reduction being more significant in the forces (reactions - P) than in saggings (y_g).

E.G., for excentrically compressed columns (twins) at a normal force N=180 t. application of powerful impulses development of plastic deformations resulted in the following reduction of dynamic coefficients: in saggings $K_g = \frac{y_g}{y_c} = 1{,}53$; in forces $K_g = \frac{P}{P_g} = 1{,}32$; at a dynamic rigidity reduced by 1.33 times ($\frac{K_1}{K_i} = \frac{T_i^2}{T_1^2} = 0{,}66$) and an over 10-fold increase of the energy absorption coefficient at powerful impulses (ψ_0=0.12 to ψ_1 =1.28).

Destruction of these columns occurred at $P_{dem} = 2{,}3\,P_{cal}$ at a maximum sag $y_{dem} = 11{,}0\text{ cm}$ $(\approx 6\, y_{cal})$

For columns with no compression (N=0) a ten-fold increase of P was accompanied by a 15-times increase of the saggings, while $K_g = 1{,}1$

Significant results have been obtained when testing a three-storey double-span (l=6.0 m.) reinforced-concrete natural-size fragment (column section 40 x 50 cm.) composed of cross-type units and simulating the 3 lower storeys of the middle frame in a 9-storey frame building (with estimated seismic resistance of 8 points). Subjecting the fragment to multiple horizontal increasing impulses (at $\tau = T/2$) and to static experimental stresses caused by the developed elastic-plastic deformation results in the dynamic coefficient dropping from $K_g = 1.95$ to $K_g = 1.43$. Destruction of the fragment occured at $P_{dem} = 86^{d} t$ ($\approx 2.15 P_{cal}$) along an oblique section of a top-storey middle column at a transversally applied strength Q=48t. ($2.4\,Q$) at an increased energy coefficient up to $\psi = 2$ with the maximum saggings attaining $y_{max} = 18.0$ cm. ($\approx 8\, y_p$). A short-duration coefficient $m_{kp} = 1.2$ has been assumed in compliance with computational standards for eccentrically compressed ferro-concrete columns and has been experimentally confirmed as regards the currently expanded energy-consuming capacity of the columns. When developing the elastic-plastic deformations and the column operation in the second boundary state, beside $m_{kp} = 1.2$ one can introduce a non-linearity coefficient $m_{n\ell} = 1.2$ to produce calculation of the frame columns for the actual-to-elevated seismic actions according to the accelerogramms.

Research by prof. I.L.Korchinsky who has suggested evaluation of the bearing capacity of the reinforced concrete structure under seismic influences to be made from an energy standpoint, has shown the elevated seismic accelerations causing residual structural deformations $\ddot{U}_p > 2\ddot{U}_{av}$ to constitute 5 to 15 per cent of the total number of loading cycles $> \ddot{U}_{av}$ with their number hitting an average of 9% ≈ 0.1 for overloading accelerograms. It is to be added that of these peak accelerations with the multiple (equal) periods $T/4 \div T/2$ (where T is the period of the building's free oscillations) the ones that can really cause plastic and residual deformations will constitute less than 9 per cent. Introducing $m_{kp} = 1.2$ beside $m_{kp} = 1.2$ into the calculations for eccentrically compressed columns results in retaining a considerable reserve of their energy-absorbing capacity. The above statement is true for the specified columns, designed and created according to the required standards. Violating these requirements, particularly on conducting structural work and on placing transversal reinforcement of columns precludes development of their elastic and plastic deformations endangering the structure at seismic overloading.

Experimentally substantiated conclusions of the author: a double-span fragment of prefabricated monolithic frame (with column sections 50 x 50 cm.) of the middle section of a 14-storey framework simulating the 7-8 storeys with the closed collars being replaced by loop outlets at 30-cm. pitch and also in supporting zones when testing for the above-mentioned short-time static and dynamic loading has shown fragile failure by segregation of the left column and under a steep oblique section also of the right one, the transversal force for each column being Q=27-30 t. (≈ 1.35 to $1.5\, P_{dem}$) with no compulsory appearance of plastic deformations. Similar failures of ferro-concrete columns have been observed at powerful earthquakes in Chile, Bucharest, etc.