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PERFORMANCE OF REINFORCED CONCRETE BEAMS UNDER SEISMIC AND CYCLIC LOADINGS

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SUMMARY

flexure and shear are assessed. Axially compressed concrete, reinforcement in air and reinforced concrete elements under axial tension were additionally analysed in order to get a complete picture of the cyclic capacities and performance of materials which constitute reinforced concrete beams. Microcracking, onset and development of macrocracks, accumulation of strains and stresses, change of asymmetry coefficient, type of reinforcement, shear span, shear reinforcement ratio depending on the load characteristics and number of loadings are considered. Determining of strains in all components at any particular number of cyclic loadings opens a way to the performance based analysis and desig

INTRODUCTION

during earthquakes is not addressed. An exact performance is a rather soft target. An acceptable overall behaviour is supposed to be guaranteed by the prescriptive detailing, but not by analysis. At the same time the seismic design, unlike the design for gravitational loads, allows substantial post-elastic deformations. Actually, current methods of design for single event and single performance level objective of collapse prevention implies that a high degree of damage-just short of collapse- is accepted. So, the deformation demands and capacities are to be defined securely and located accurately. Development of cracks, accumulation of residual strains, stress redistribution, stiffness degradation, change of response and reduction of strength are all taking place simultaneously. They are virtually inseparable. Presently employed force reduction factors depending on the available ductility, cannot do justice to the problem. The initial data needed for the performance based analysis are not adequate. The fundamental expression of a structural response can be obtained on the basis of strains. Both the performance and strength criteria can be formulated on the common footing. This study was initiated in order to obtain the basic experimental data for the materials separately and after that when they are combined together in a form of the beams subjected to flexural and shearing action

TEST PARTICULARS

air, reinforced concrete elements under axial tension, traditionally reinforced concrete beams, partially unbonded beams and beams reinforced by external steel strips were investigated. Variables: levels of stresses and their asymmetry coefficients, frequency of loadings, number of cycles, concrete grade, type and grade of longitudinal reinforcement, shear span, shear reinforcement ratio, location and length of unbonded part, local reinforcement of concrete compressed zone. Strains of concrete and reinforcement and deflections of beams were measured directly during cyclic loading and periodically by static tests after specific numbers of cycles. Periodic static testings included measurements of cracks development, speed of ultrasonic waves propagation, assessment of Poisson's ratio and change of volume. All beams were of the same cross-sectional dimensions and longitudinal reinforcement ratio, namely, 100*220*2100mm and r_s = 2%. Concrete grade varied from C30 to C50. Shear span varied from 1.74 to 2.70. Shear reinforcement ratio varied from 0.0 to 0.6%. Local reinforcement in a form of welded meshes placed over the potential flexural crack was used. Ultrasonic wave apparatus was used for the determination of the onset of micro-cracks in concrete. Frequency of loadings was within the range of

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300 - 750 cycles per minute. Asymmetry coefficients of externally applied loadings varied from 0.0 to 0.70. Beams tested in shear were made equally strong in static shear and static flexur

CONCRETE

concrete. In spite of numerous investigations a clear understanding of the physical phenomenon is still elusive. Since all previously existing methods of analysis were concerned with the ultimate state only, the factual

$$f^{0}_{crc} / f_{pr} = 0.35(\log f_{pr} + 1) - 0.5 \qquad (1)$$

performance at all intermediate states was largely overlooked and investigations were concentrated on the load bearing capacity. Microcracks develop at relatively low levels of stresses. They signal a beginning of a gradual failure of the weakest components of concrete. The further response and behaviour are heavily influenced by this early microcracking. After the onset of microcracks a concrete cannot be considered as a continuous medium. Theories of elasticity, plasticity and relaxation do not apply in their classic forms. For the concrete grades C40 - C50 it was found that the bottom level of microcracking can be reliably determined by the method proposed by O. Berg.

The upper level of microcracking was found to be in average 1.6 times larger than the bottom level.

Stress-strain curve (Fig. 1) is continuously departing from the maiden curve and gradually changing its curvature direction with the number of cyclic loadings. The change of curvature signals a fatigue distress and is regarded as a precursor of the fatigue destruction. Under high levels of stresses the development of cyclic creep is very rapid. The first loading can result in 25 - 35% of the total creep of the concrete before the collapse. Within the

$$\Delta\varepsilon_{c\,N} = (N / f)^{1/3}(\gamma_{\max} + \gamma_{\min})(1 - \rho^2)(1 + 8(\gamma_{\max} - \gamma_{\min})) * 10^{-5} \qquad (2)$$

low-cycle region the increase of cyclic creep can be defined as:

where f is the frequency in hertz, $\gamma_{\max} and \gamma_{\min}$ are respectively the maximum and minimum stress levels of a loading cycle relative to the static strength, $\rho = \sigma_{\min} / \sigma_{\max}$ is the stress asymmetry coefficient. In high-cycle

$$\varepsilon_{c,N} = \varepsilon_c[1 + 1.1(1 - \rho + \rho^2)(1 - 0.5\gamma_{\max})(1 - e^{-0.25\log N}) \qquad (3)$$

domain the cyclic creep of axially compressed concrete may be determined as follows:

Modulus of elasticity is substantially decreasing with number of cyclic loadings (Fig. 2). The reduced value of elastic modulus at any number of cycles can be defined in the following way:

$$E_{c,n} = Ec[1 - 0.07(1 - \rho)\log N \qquad (4)$$

Although there is a general agreement about the reduction of elasticity modulus the magnitude and the rate of this decrease are not defined clearly. This process depends on a variety of factors and is subject to considerable variations. Under common conditions the decrease may be expected to be (45 - 50)%.

There are two clearly recognisable regions in concrete response: the low-cycle zone and the high-cycle domain (Fig. 3). Experimental findings yield the following evaluation of fatique limits.

$$f_{c,f} / f_{pr} = 1.21 - 0.18\log N, and, f_{c,f} / f_{pr} = 0.97 - 0.064\log N \qquad (5)$$

The intersection point between two regions is at N-117. Within the low-cycle region an influence of the number of cycles is considerably stronger than that in the high-cycle zone. From these results a very important conclusion emerges. Seismic strengthening of (30 - 50)% adopted in the majority of current codes of practice do not have unquestionable experimental backing. When $N = 2 * 10^6$ is adopted as an assessment basis, the most likely estimate of high-cycle fatigue under asymmetry coefficient ρ=0 was found to be $f_{c,f(\rho=0)} = 1.4 f^{0}_{crc}$..(6)

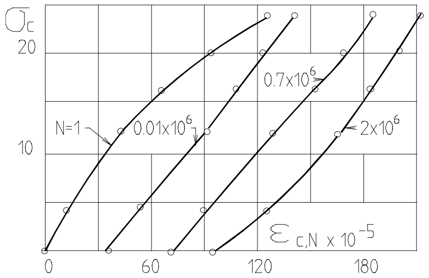


Fig.1 Stress-strain relationship for concrete

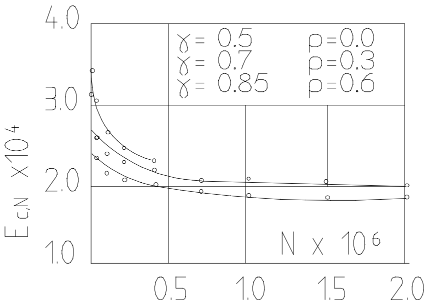


Fig.2 Concrete modulus of elasticity

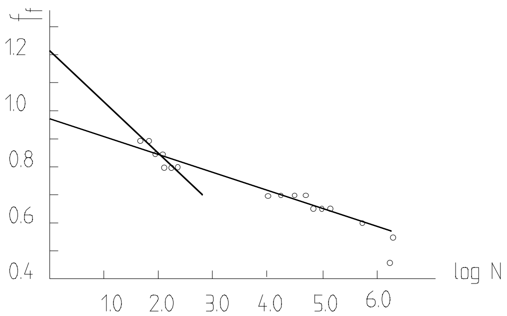


Fig.3 fatigue of axially compressed concrete

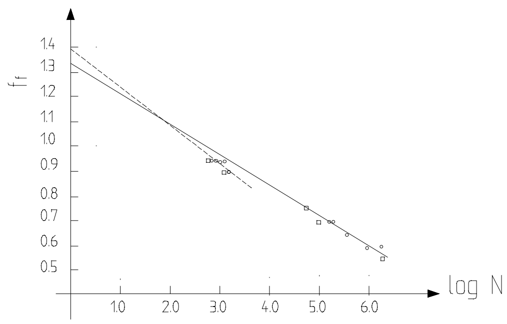


fig.4 fatigue of reinforcement

By stating this we are siding with the concept that a relative fatigue limit increases with concrete grade. It is a highly controversial point which needs further investigations. It contradicts with the fundamental reasoning that more brittle materials have lower relative fatigue limits. Since ductility of concrete is decreasing with its grade increase, this signals a real trouble. This finding cannot be stretched to the low-cycle region. Also it is relevant only to the limited concrete grade diapason, namely, C40 - C50. At present there are diametrically opposite evidences about the fatigue limit - strength of concrete relationship. No convincing explanation has been offered for these striking differences. In this study the high-cycle fatigue limit under ? = 0 was estimated to be

$$f_f = f_{sust} - (f_{sust} - f_{f(\rho=0)})(1-\rho^2)\text{..(7)}$$

where f_{sust} is a sustained static strength, when ρ = 1.0. In most cases can be adopted as 0.85 f_{pr}. For C40 - C50 grades a well substantiated experimental average of 1.4 $f^{0}_{crc} = 0.5 f_{pr}$ can be used, and then

$$f_{c,f} / f_{pr} = (0.85 - 0.35(1-\rho^2)\text{..(8)}$$

In this analysis the sustained static strength is treated as a particular case of a cyclic loading with ? = 1.0. When N=1 the cyclic loading transforms into a dynamic, impact type, actio

REINFORCEMENT IN AIR AND R.C. ELEMENTS UNDER AXIAL TENSION

$(a) f_y = 470, f_u = 687, E_s = 1.98x10^5, (b) f_y = 320, f_u = 545, E_s = 2.04x10^5, (c) f_y = 441, f_u = 687, E_s = 2.06x10^5$

These experiments have conclusively demonstrated a reduction of fatigue capacities of embedded reinforcement as compared to that in air. Low-cycle fatigue limit of reinforcement in air and in tensile elements are:

$$f_{s,f} / f_y = 1.39 - 0.144 \log N, and f_{s,f} / f_y = 1.35 - 0.15 \log N \ldots\ldots\ldots\ldots(9)$$

The difference between them is small but one-sided. High-cycle fatigue limits of reinforcement in air and that in axial tension r.c. elements were respectively defined as:

$$f_{s,f} / f_y = 1.335 - 0.119 \log N, and f_{s,f} / f_y = 1.31 - 0.118 \log N \ldots\ldots\ldots\ldots(10)$$

These tests did not show any considerable difference between low-and-high-cycle fatigue capacities (Fig. 4). The other very important outcome is that within both the low-cycle (100 - 200 cycles) and the high-cycle regions there were no necking-localised reduction of diameter of specimens and no sudden extension or discontinuous yieldin

BEAMS IN FLEXURE

standard static conditions. Spacing between flexural cracks was found to be practically the same as that at the static service level of loading (i.e. 0.67 of the ultimate moment), when the number of cracks was already stabilised. Stabilised spacing between cracks can be assessed by:

$$l_{crc} = ((h - x)/x)(cA)^{1/3} E_s * 10^{-5} \ldots\ldots\ldots\ldots(11)$$

where c=distance from the tensile face to the centroid of longitudinal reinforcement. A=average effective area of concrete in tension around each bar. A=2*C*b/ number of bars. It is of interest that irrespective of load characteristics there was a limiting crack depth. As a result the compressed zone depth before the collapse under all load characteristics was practically the same. Crack's development was very rapid during the initial cyclic loadings. The cracks width can be defined by:

$$w_N = l_{crc} * \varepsilon_{s,N} \ldots\ldots\ldots\ldots(12)$$

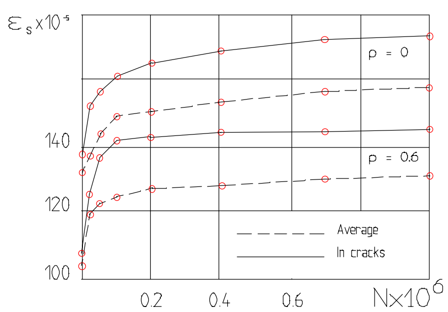


Fig.5 Strains in longitudinal reinforcement

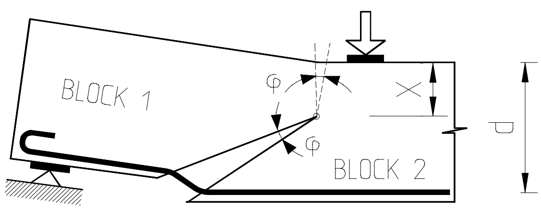


Fig.6 Conventional rotation of blocks

In more accurate assessment the effect of reinforcement stiffening by concrete between cracks can be taken into consideration. The ratio between the maximum and average width of cracks was found to be 1.65, which is in an agreement with the majority of experimental works. Generally the width of cracks is strongly related to the strains in reinforcement, and for this reason the key to this problem is in the reliable assessment of strains.

The strains of compressed zone of concrete are continuously increasing with number of cyclic loadings. Cyclic creep of concrete in low-and-high-cycle regions can be respectively taken as:

$$\varepsilon_{c,N} = (N/f)^{1/4} 15(\gamma^2{}_{\max} - \gamma^2{}_{\min})(1-\rho^2)10^{-5} \text{...} (13)$$

$$\varepsilon_{c,N} / \varepsilon_{c,el} = 1 + (1-\rho+\rho^2)(1-0.5\gamma_{\min})(1-e^{-0.35\log N}) \text{..} (14)$$

Ultimate strains of concrete were generally lower than those under static conditions. When load parameters permit to apply 2*106 cycles of loadings there was a tendency for convergence. In beams with partially unbonded reinforcement the local strains were much larger and there were cases of brittle, explosive type fatigue failure of compressed zone of concrete.

There was a large increase of residual stresses in longitudinal reinforcement with number of cyclic loadings (Fig. 5). Depending on load parameters and number of cycles the increase was as large as 100 - 130 Mpa. Accumulation of stresses can be very rapid. In some of our tests the first loading resulted in (25 - 55)% of the total residual strains developed due to cyclic loadings. As a result beams suffered the early distress. The increase of strains was observed to be:

$$\varepsilon_{s,N} / \varepsilon_{s,el} = 1 + (1-\rho+\rho^2)(1-0.5\gamma_{\max})(1-e^{-0.25\log N}) \text{..} (15)$$

An increase of strains is accompanied by concurrent increase of their asymmetry coefficients. For low-cycle region the increase was estimated to be:

$$\Delta\rho_s = 0.2(1-\rho)(1+0.5\gamma_{\max})(1-e^{-0.75\log N}), and \rho_{s,N} = \rho_{s,1} + \Delta\rho_s \text{.................................} (16)$$

The location of a fatigue fracture in tensile reinforcement within zero-shear zone was predetermined by bond. Fatigue ruptures took place not directly across flexural cracks but at a distance of 1 - 2 cm from the edge of cracks towards uncracked concrete. The point of maximum stress concentration is shifted to the section where there is still a bond. Fatigue fracture of deformed reinforcement initiates at the intersection of longitudinal and lateral ribs. Performance of externally placed steel strips was heavily affected by electric welds. Reduction of fatigue limit was within (33 - 65)%.

When strains in tensile reinforcement and compressed concrete are known the deflections of beams can be easily determined. An increase of deflections was observed to be within the range of 1.5 - 2.0 times a value at the first loading. The residual component was approximately 2.7 times larger than the elastic one. It is well below the ductility demand of 4 which is commonly adopted as a minimum in seismic design. But in our case the increase of deflections was not due to yielding of reinforcement. It was a result of cyclic creep of concrete, development of cracks and partial deterioration of bond. A deflection recovery after the removal of loading is gradually diminishing from approximately 75% to 40

BEAMS UNDER SHEAR

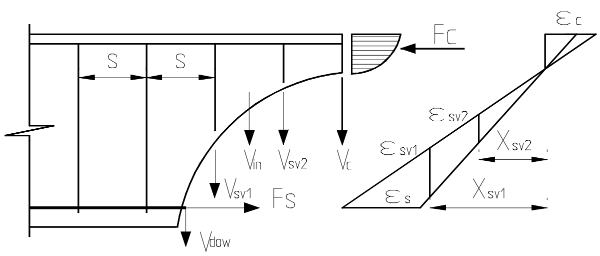
development of inclined cracks length terminates and the further increase of external loads is accompanied only by an increase of a width of inclined cracks. The principle of mutual rotations proposed by R. Walther is clearly visible (Fig. 6).

Under static conditions beams were equally strong in both the flexure and the shear. Six out of 10 beams failed in flexure and remaining four-in shear. But under cyclic loadings all beams failed in shear. Actual stress-strain conditions along an inclined crack cannot be readily assessed. Some rough assumptions which are in dangerous conflict with reality are to be adopted. In our case an extremely complicated actual strain condition is reduced to a linear relationship (Fig. 7). Normal and shear stresses distribution across the compressed zone is continuously changing. A simplified relationship between normal stresses and shear force is shown in Fig. 8. These stresses cannot be evaluated accurately. Residual creep component and relaxation of stresses do not allow to define an exact stress-strain relationship. Asymmetry coefficients of compressed zone are changing with number of cycles. Moreover, they are different over the compressed zone depth. Only stirrups crossed by inclined cracks experience sharp increase in strains. The largest strains are registered at the base of inclined cracks. The increase of strains, the stresses and asymmetry coefficients in stirrups can be respectively defined as:

$$\Delta\varepsilon_{sv,N} = (3.5 - 3\rho)(\gamma_c /(1-\gamma_c))\log N 10^{-5}, where \gamma_c = \sigma_c / f_{pr} \text{..} (17)$$

$$\sigma_{sv,N} = \sigma_{sv,1} + \Delta\sigma_{sv,N} \text{..} (18)$$

$$\rho_{sv} = \rho + (0.2 - 0.2\rho) \text{..} (19)$$



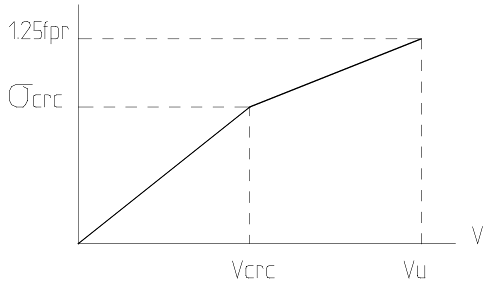
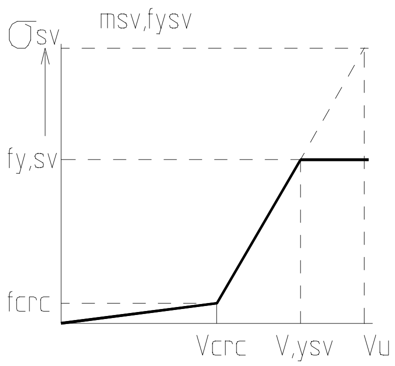


Fig.7 Shear forces and strains distributions **Fig.8 Normal stresses and shear forces**



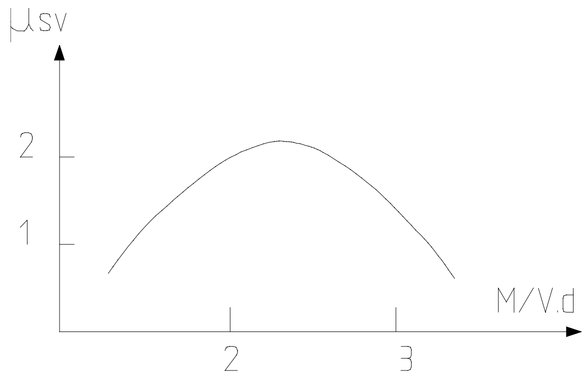


Fig 9 Shear forces vs. stirrup stresses **Fig.10 Shear span vs. shear ductility**

Stresses in stirrups and the available ductility can be approximated by the diagram shown in Fig. 9. Post-elastic ductile response is related to M/Vd, which ultimately separates the shear and flexural responses. Shear ductility depends on the shear span and both the shear and longitudinal reinforcement ratios (Fig. 10). For $r_{sv} = 0.4\%, and, r_s = (2-3)\%$ the ductility was found to be:

$\mu_{sv} = 5.84(M/V*d) - 1.27(M/V*d)^2 - 4.5$...(20) Shear ductility is basically attributed to shear reinforcement. But the increase of shear reinforcement ratio beyond 0.7% results in the reduction of ductility and subsequently in a change of the mode of failure. For M/Vd=2.5 - 3.0 the fatigue of stirrups can be expected when $r_{sv} < 0.6\%$ and fatigue of concrete when $r_{sv} > 0.7\%$. At high levels of loads shear cracks play a considerable role in the increase of deflections. At the service levels of loadings (M=0.67MU) this effect can be roughly taken as contributing 15% of the total deflection. At higher levels of cyclic loadings it can constitute a much larger part.

A very spectacular behaviour was exhibited by beams without flexural bond over the shear span. Shear failure became a very remote possibility. Strains were concentrated in a singular flexural crack at the end of unbonded shear span. Reinforcement acts as a tie-bar and stresses in it remain permanent in spite of the fact that there is a change of the external bending moment. An angle of local rotation can be defined by using deformations in reinforcement.

$\varphi_{max} = 9\Delta l_{max} / 8d$ where (8/9)d is the lever arm. …………………………………………….(21)

Longitudinal cracks which split off a beam into a compressed and tensile zones were formed at load levels of 0.75 - 0.85 of ultimate values. Local reinforcement of compressed block by steel meshes has proved to be very effective. The capacity of compressed zone can be increased, its deformations-restricted, crack opening-arrested and the mode of failure-change

CONCLUSIONS

indication of their behaviour in beams. Also there are two clearly visible regions in concrete response-low-and-high-cycle domains-the process is basically continuous. Within low-cycle zone the frequency of loading and number of cycles are strong variables. Microcracking of concrete is a definite precursor of the fatigue distress. Reinforcement tested did not exhibit any considerable difference between low-and-high-cycle fatigue regions. Within both zones there were no necking, sudden extension and discontinuous yielding. Fatigue fracture originates at the intersections of longitudinal and lateral ribs. The dynamic strengthening of both concrete and reinforcement was found to be much lower than that commonly adopted in seismic design. Ultimate strains under cyclic loadings were generally lower than under static conditions, but when load parameters allow to apply two million cycles there is a tendency for their convergence.

Performance and capacities of embedded reinforcement is radically different from those of the free reinforcement. Cyclic creep of compressed concrete, development of cracks and partial loss of bond result in the continuous accumulation of residual stresses in reinforcement of beams. An increase of stresses can be as large as 100 - 130 Mpa. Moreover the stress asymmetry coefficients are changing concurrently with stresses. So, the factual stress-strain condition is totally different from that experienced at the first loading and simply not comparable to that of reinforcement tested in air.

Both the flexural and shear cyclic crack resistances are approximately 50% lower than those under static loadings. Crack width and deflections may be expected to be 2 times larger than those due to the first loading. Within the low-cycle region beams can suffer a very rapid loss of stiffness and a sharp distress.

Shear ductility first of all depends on the physical ability of stirrups to yield, on the shear span and shear reinforcement ratio. Ductility is decreasing when the shear reinforcement ratio is larger than 0.7%. This limiting value separates two modes of shear failure. Beams which under static conditions were equally strong in both flexure and shear have all invariably failed in shear under cyclic loadings. Cyclic shear is much more demanding and less predictable than the static one.

Fatigue capacities of externally located steel strip reinforcement were heavily affected by electric welds. Fatigue cracks always start at the welds and the reduction of fatigue limit can be as large as 65

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