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Capacity of Damaged Reinforced Concrete Beams

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The monograph resents the results of experimental and theoretical research of the features of the work of reinforced concrete T-beams, damaged in the process of operation, and proposed a technique for determining their residual load capacity, which takes into account the actual stress-strain state and based on the basic assumptions of the current regulatory documents. Proposal shave been developed to take in to account the size and nature of damage in determining the load-bearing capacity of such elements.

For scientific and engineering staff of designnand construction organizations, graduate students, masters, students.

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CONTENTS

INTRODUCTION

One of the most essential parts of the country's national wealth is its fixed assets. They are constantly replenished and updated. However, the economic efficiency of the economic complex depends significantly on the quality of the technical operation of the fixed assets, which include also the buildings and facilities.

In connection with this, there is an acute problem of estimation of technical condition of existing building structures damaged in the operation. A reliable assessment of their bearing capacity, on the one hand, will prevent the occurrence of accidents and on the other hand to fully utilize the remaining bearing capacity of bearing elements.

The current method of calculation of reinforced concrete structures for the first group of limiting states is quite high reliability and is based on experimental and theoretical studies conducted for bent elements (in the case of oblique bending).Recommendations regarding the calculation of the bearing capacity of reinforced concrete beams bent of most general T-sections in the existing standards are absent.

Quite a wide range of experimental and theoretical studies of oblique bent concrete elements has led to the creation of methods of calculation of the bearing capacity. However, these studies considered only the intact (typically symmetrical) cross-section, and the calculated moment acted in the direction of both axes.

The work of reinforced concrete structures, in which oblique bending is realized as a result of damage (asymmetric with respect to the principal axes of the cross section), currently remains unexplored. Analysis of stress strain state and the determination of residual bearing capacity of reinforced concrete beams of t-shaped profile damaged in the process of operation, will give the opportunity to calculate these designs for the first group of limiting states.

Therefore, experimental and theoretical investigation of stress-strain state, and work, and on their basis – the creation of methods of calculation of reinforced concrete bending structures damaged in the process of operation, the results of which are summarized in this monograph will be useful both for researchers and for practical application in the design process, construction and operation of buildings and structures.

The authors express their deep gratitude to doctor of technical sciences, professor Z. Ya. Blicharskiy and Ph. D., associate Professor O. A. Dovzhenko for their useful suggestions and advice on the structure and content of the book.

CHAPTER 1.

THE STATE OF STUDIES OF THE RESIDUAL BEARING CAPACITY OF DAMAGED BENDING REINFORCE CONCRETE ELEMENTS

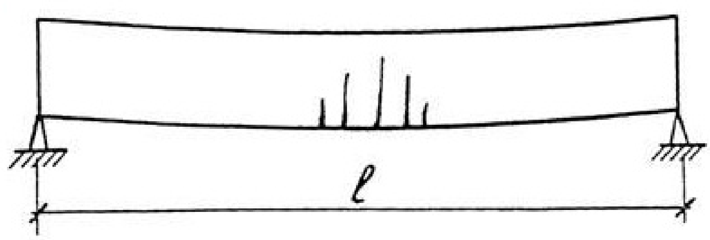
1.1. Research of work of reinforced concrete beams

Reinforced concrete has been, is and will be for the foreseeable future, one of the basic building materials. The predominant share of reinforced concrete constructions works in bending. The main bearing elements of these systems are beams.

Reinforced concrete elements of T-section are used in the buildingboth as separate structures - typical beams, and as parts of overlaps – monolithic ribbed and prefabricated panels.By the arrangement of shelves in the compressed zone the height of the beams under the ceiling, and the weight of the structure decreases, making them more efficient and convenient for use in the construction of public buildings, shopping and entertainment venues.

If we consider a reinforced concrete beam lying on two supports, in its migration from the existing loads occur normal and (or) inclined cracks (Fig.1.1,a), contributing to the destruction of the item.

a)



b)

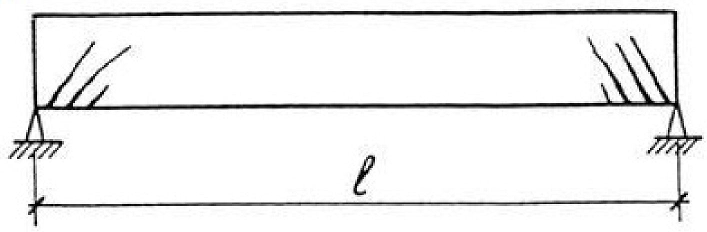


Fig.1.1. Cracks in concrete beam: a – normal; b – inclined.

To prevent the destruction of beams in the zone where the concrete is poor (lower zone − a zone of tension) steel bars are placed. Normal section (1-1 in Fig. 1.2) is perpendicular relative to the axis of the element and perpendicular to the working reinforcement (Fig.1.2). The inclined cross-section (2-2) is at an angle to the axis of the element.

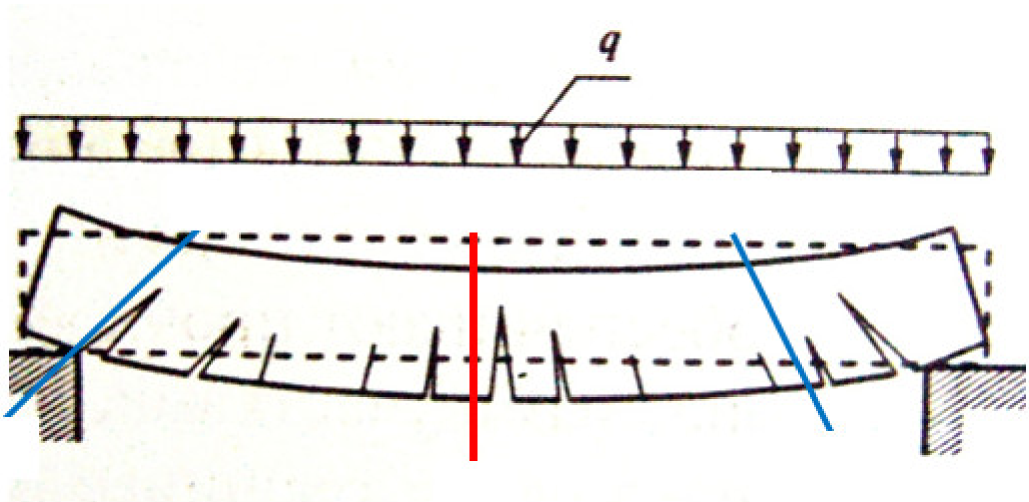


Fig. 1.2.The appearance of cracks in the concrete element.

In order to protect the element from being destroyed along the inclined sections it is reinforced by special reinforcement products (limbs, individual rods, cages), which as well as the main operating reinforcement are arranged perpendicular to the axis of the considered section.Most dangerous sections, on which inclined cracks are formed, are areas close to the poles (≈1/4 of the beam length).

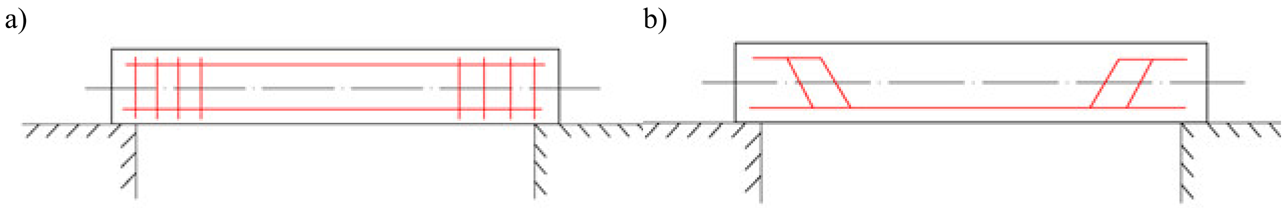


Fig. 13. Types of reinforcing elements to protect the beams from the inclined cracks: a – gaining of element by vertical rods; b - the same with limbs.

The destruction of the reinforced concrete item begins when at its most dangerous sections the concrete reaches the limit values or the reinforcementgets the limiting deformation.The destruction begins in the sections in which there are maximum values of the stresses in the material.

The destruction of the item at oblique and normal cross-section is derived from various influences.In the normal cross-section the construction is destroyed by the action of bending moment (M), and by an inclined cross-section of the combined effects of shear forces (Q) and bending moments (M).

In reinforced concrete non-reinforced bent items destruction begins with the stretched zone, when the calculated resistance of stretched reinforcement reaches its limit. The occurrence of stress in the compressed zone of concrete and the destruction of the protective layer causes destruction of the underlying layers and the gradual stretching of the longitudinal reinforcement. The increased deflection of the element causes cracking in the protective layer of concrete in the stretched zone. However, if the item is reinforced, the destruction begins from the compressed zone of concrete; the stress in the stretched reinforcement will be below the limit values, which does not meet the requirements of optimal design of items. The destruction of the reinforced concrete element of the compressed zone may not only be due to incorrect design. The destruction of the compressed zone of concrete also takes place at low temperatures of heating and large compressive stresses in it, when the plastic properties of concrete at heating does not have time to occur and the modulus of elasticity is reduced slightly. This case is similar to the destruction of reinforced concrete bending elements at normal temperature and is characterized by the underutilization of mechanical properties of stretched reinforcement.

Thus, there are two schemes of destruction of reinforced concrete "normally" reinforced bending elements:

1) when the cause of the loss of strength of the item is reaching in the stretched reinforcement calculated resistance (Rs) of yield strength;

2) when the strength of the item is exhausted due to the destruction of the compressed concrete zone before the stress in the stretched reinforcement reaches the design strength.

When calculating the strength of normal section of bending items it is assumed that in the stretched zone of concrete cracks appear, and concrete in this area no longer works on the perception of stress. Given that there is a need to optimize the size of building structures and reduce their consumption of materials, try to minimize the consumption of materials (in this case, the concrete in the stretched zone), without compromising the strength of the item as a whole.

In a rectangular beam section, you notice that the lateral portions of the stretched area do not participate in the load bearing capacity; therefore, they can be removed. Thus, it appears T-section of beam. Such beam of reinforced concrete works due to the fact that compressive forces are taken up by a concrete shelf (and, partly, by an edge), and tensile forces by a sufficient amount of reinforcement in the edge of the item.

Applicable rules [68] do not include the possibility of determining the residual bearing capacity of damaged reinforced concrete beams of T-section, although it could significantly reduce the cost of strengthening, and the study of stress-strain state of such structures would allow to analyze them in future work together with the construction of strengthening.On this basis, relevant is the assessment of the technical condition of damaged reinforced concrete bending element, i.e. the estimation of the residual bearing capacity, which this construction has at a certain moment of operation. Further by comparing the residual carrying capacity with the magnitude of external influence it is possible to evaluate the technical condition of the construction and the possibility of its further operation. Since designing is carried out by the method of limit States, reserves of strength in the process of wear is not provided, and the slightest damage (reducing the cross-sectional area) immediately turns the design into unserviceable condition [59].

Analysis of existing normative documents to determine the technical state of constructions [80, 106] showed that they also contain recommendations for determining residual bearing capacity of constructions.

The study of damage of reinforced concrete structures from exposure of the external environment are works of domestic and foreign scientists such as: S. N. Alekseev [3], V. O. Almazov [5], D. O. Astafiev [6], V. I. Babushkin [7], A. Ya. Barashikov [9], V. M. Bondarenko [18, 19, 20], V. A. Bondar [17], Z. Ya. Blikharskiy [13, 14, 15], E. A. Guzeev [39], E. V. Klimenko [57], E. G.Pakhomova [88], A.V. Semko [100], R. S. Khmil [109], S. S. Chempion [110], B. A. Yagupov [112, 113, 114], Adhikary B.B. [117], Al-Bayati N. [118], Constantin

E. Chalioris and Constantin N. Pourzitidis [120], Hassan A. [121], J.Jayaprakash [122], Smith Roger W.[127], Wu Hao [128] and many others.

In the work of S. N. Alekseev [3] the processes and the occurrence of corrosion and protection from it are set out in detail.

Extensive research of work of reinforced concrete bent constructions under the joint influence of load and aggressive environment were held by A. Ya. Barashikov and Z.Ya. Blikharsky Z.Ya. [9, 13, 14, 15].They found that even among the 10% of acid mainly the outer layers of concrete are damaged. The lower layers generally have sufficient strength and with a corresponding strengthening (or justification for further normal operation without special reinforcement) can be used in the future. Assessment of residual bearing capacity was studied in the works [18, 19, 20, 112, 113, 114]. The authors provide recommendations for determining residual life of constructions with regard to the development of creep strain and the kinetics of damage accumulation in time.

The monograph of S. S. Chempion [110] shows the classification of effects on building constructions, namely: corrosion, erosion, biological effects.

In the work of Smith Roger W. [127] experimental studies of corrosion effects on the structural behavior of reinforced concrete beams was carried out. The results of experiment of testing beams showed that the bearing capacity decreases with increase in the area of corrosion and showed gradual strength loss due to increase in the area of corrosion.

Hassan A. studied the effect of corrosion on strength, used the samples with four different types of concrete and three different types of steel. Foreign experience [121] shows that the strength for corrosion reinforcement and stainless steel was lower than that of the reinforcement of the permanent profile of carbon steel. At low level of corrosion (about 0.5 to 1% of the total mass), there is a slight decrease in strength. The strength decreases with increasing corrosion level for all sample types.

The development of corrosion in damaged reinforced concrete bent beams is considered in the work of V. A. Bondar V. A. [17]. He proposed a method of forecasting of this process and description of its impact on residual bearing capacity.

The issue of the development of biological corrosion and its influence on the occurrence of damage to building constructions is studied in the monograph of E. Grunau [36]. Such corrosion is observed mainly in organic building materials or constructions on their base.

In the work of A. I Popesko [92] for evaluating the performance of reinforced concrete elements subject to corrosion, theoretical and experimental and theoretical aspects of the nonlinear theory of calculation of reinforced concrete structures were worked out. The proposed theory of calculation was developed on the basis of the existing provisions and hypotheses the nonlinear theory of reinforced concrete and existing concepts about the mechanism of corrosion of concrete and reinforced concrete. In addition, in the work there was obtained analytical dependence between stresses and deformations at short-term compression of concrete susceptible to corrosion. There were rates proposed which take into account changes over time of strength of corrosive concrete in the most characteristic of aggressive environments.

O. V. Stepova [104] developed a method which allowed obtaining the necessary parameters for the calculation by non-destructive method while inspecting the construction in operation, and predicting the loss of section area of reinforcement in the crack, in the particular mode of operation.

Analysis of the technical state of reinforced concrete bending elements, namely the study of defects and damages was described in the works of: D. N. Baida [8], O. I. Valova [23], E. P. Voskoboynik [30], V. T. Grozdov [38], V. S. Dorofeev [44...47], A. H. Kara Damur [51], E. V. Klimenko [59], A. I. Malganov [75], B. M. Mizernyuk [78], T. M. Petzold [89], A. A. Pischuliov [90, 91], Yu. D. Rybakov [94], I. L. Sisin [102], B. A. Yagupov [115] and many others.

Research of D. N. Baida [8] is about solving the problem of estimating the residual bearing capacity of reinforced concrete beam elements which in operation have been under the influence of destructive loads. An examination of these elements reveal damages such as bulging of the compressed reinforcement, concrete breaking, crumbling of aggregate in the compressed zone, the critical values of deformation and crack opening. The technical condition of reinforced concrete elements with such injuries or emergency is recognized inoperative or emergency. The work proposes under certain conditions to assume these elements partially destroyed. In particular the dependencies are proposed, which describe the deformation diagram of concrete in compression after gaining the ultimate strain in concrete to complete physical destruction of the concrete. Under the current model there was proposed the dependence to calculate the limiting deformations of concrete in the compressed zone of long-term loaded reinforced concrete beams. This work [36] only affects beams of rectangular cross section, and the proposed method of calculation of residual carrying capacity does not apply to bending elements of the T-shaped profile.

In her work [30] E. P. Voskoboynik has performed a typological comparison of typical defects and damages of reinforced concrete, metal and composite girder beam constructions of T-section. The author summarized that at the same parameters of damage, in some cases composite steel reinforced concrete constructions are more tenacious than similar metal and reinforced concrete, which is a significant advantage of complex constructions.

In the work of B. M. Mizernyuk [78] there were described the defects and the destruction of reinforced concrete constructions, observed in operated facilities during the examinations, selected from a large number of operations carried out by the Central laboratory of the theory of reinforced concrete of NIIZhB, and Bureau of implementation of NIIZhB, the causes of which depend on the design assumptions, except in cases of defect of production.

Yu. D. Rybakov [94] reviewed the various defects encountered in the installation of reinforced concrete precast-monolithic constructions of multistoried industrial buildings. The work presents practice-proven methods of reinforcement of constructions and corrections of defects occurring.

The authors of [102] revealed the typical damages of reinforced concrete crane beams by span of 12 m and assessed for loading capacity and crack resistance of a crane beam in the calculated cross sections of operational loads. There was proposed a construction of strengthening of crane beams.

The research by A. K. Salekh [98] discusses the study of the work of bent reinforced concrete elements, which had from one to three layers of weakened concrete in the compressed zone with different qualities, the strength of these layers varied between 16.3 MPa to 30.7 MPa. Thus, the samples were models of constructions in which physical heterogeneity of the compressed zone was unknown.

After testing, there is a tendency to "drop" stresses of the extreme fibers on a firmer layers located closer to the neutral axis of the sample. A similar process was observed in the extreme fibers at testing beams with uniform cross-section, but to a lesser extent.

The author suggests a methodology and calculating formulas for determining the resistance of cross-sections, normal to the longitudinal axis of the bending items, with the physical heterogeneity of the structure of the compressed zone. Based on the analysis of the stress-strain state of constructions after long-term operation, as well as experimentally established features of the operation of constructions after strengthening, there was proposed a method of determining the height of the compressed zone of reinforced by a compressed or stretched area of the bending items.

The testing of damaged reinforced concrete beams of T-section is also described in the works of T. N. Azizov [1, 2, 35]. The work presents results of experimental studies of reinforced concrete items of the T-section with artificially created cracks during normal pure torsion, to determine the stiffness and strength characteristics. In the factory the18 reinforced concrete elements of T-section were made, which had three normal artificial cracks, the distance between the cracks was 250mm.The cracks penetrated only the edge of the sample and were imitated by inserts fiberboard 4 mm thick. The experimental samples had the following characteristics: length – 1200 mm, total height of the cross section -250 mm, the width of the cross section of the edge – 125 mm, shelf thickness - 50 and 75 mm, shelf width – 250, 350 and 450 mm. The reinforcement of the edge was performed by single longitudinal reinforcement of class A 500 with a diameter of 12, 18 and 22 mm; reference area was reinforced by closed clamps; shelf by grid located at the middle of its thickness. As a result of theoretical and experimental studies there was developed a method of calculating the strength of T-item with normal cracks in torsion with unreinforced and reinforced shelf. It was found that the T- shaped items with normal cracks can be destroyed in three schemes: from the edge of the compressed zone of concrete under the action of becket force of reinforcement,of the destruction of the compressed zone of bending, as a result of an external torque, as a result of the puncturing of the longitudinal reinforcement. It was also experimentally found that when increasing the diameter of the longitudinal reinforcement, width and height of the shelf, the stiffness of an item with normal cracks increases.

In [1, 2] raised the study of damage to the edges of the beam, but there is no research on the damage to the beam shelf and the influence of these factors on the residual bearing capacity of the latter.

The work of P. S. Homon [35] describes a nature of the destruction of T-shaped beams from the action of repeated loads. The criterion of destruction of the beams was reaching the relative deformations of reinforced concrete and reinforcement of limit values. The destruction of all reinforced concrete beams of T-shaped cross-section held in the compressed zone, by detachment with further shearing. It was found that the low cycle load due to compaction of the compressed zone of concrete increases the bearing capacity of reinforced concrete T-section items: for the level of loading of the (0 ... 0,6) η by an average of 6%, for the level of loading (0 ... 0.75) η to 6 ... 9%.

Of special note is the work of V. N. Alekseenko [4], in which the technique of research is substantiated, allowing to establish regularities of change of the residual bearing capacity of reinforced concrete bending items under the action of shear forces after the fire.

Increasing of deformability, reducing of crack resistance and bearing capacity of the beams after the fire by the action of shear forces is due not only to a decrease in the strength and deformation characteristics of concrete and reinforcement, but also a change in the conditions of their work together.

Analysis of variance of the results of experimental studies have revealed a dominant influence of the heating temperature of the transverse reinforcement with the concrete to reduce the strength of beams on inclined sections under the action of shear forces after the fire. The intensity of the transverse reinforcement, shear span, height of section beams do not make a tangible contribution to the change in the relative strength of the beams on the sloped sections under the action of shear forces after the fire. The dependence of the strength of oblique section transverse force from the above factors for non-heated beams retain their qualitative and quantitative characteristics for beams after the fire as well.

Not the least role in the occurrence of defects and their impact on the bearing capacity of reinforced concrete bent elements plays the technological damage of concrete. A team of leading scientists have worked in this direction, including the achievement of which we should mention the study of the Odessa scientific school V.S. Dorofeev and his disciples [44 ... 47]. In their works they reflected the problems of the heterogeneity of concrete, its damage by the initial defects, the impact of these defects on the further work of material deformation, cracking and fracture, as well as the bearing capacity of bent reinforced concrete items. In work it is proved that there is a possibility of control of technological damage of concrete due to changes in the number and dispersion of the filler, and the production technology of the material.

For example, in the work of V. S. Dorofeev and N. V. Oleinik [83] the stress-strain state of bending reinforced concrete items was analyzed, which showed that at the initial stage of development normal cracks, as well as sloping, are developing by the energy-efficient path, which is trajectories of technological cracks. Therefore, by controlling technological damage, it is possible to change the working conditions, the kinetics of growth and, partially, the trajectory of the cracks.

Issues of durability of building constructions and the technical state of buildings and structures in general were considered in the works of M. V. Zavoloka [48], E. V. Klimenko [60...63], P. I. Krivosheev [70], V. V. Matveev [76], N. V. Savitsky [97] and other scientists.

In the works of E. V. Klimenko [60...63] there was proposed a methodology for the identification, prediction and control of technical condition of individual constructions and of buildings and structures. In particular, these works consider the use of damaged reinforced concrete bending and compresed items, provide guidance on how to assess residual bearing capacity. However, in these studies the damage considered, the front of which is parallel to one of main axes.

N. V. Savitsky [97] provides the general analysis of the reasons that most often cause the failure of constructions. In his opinion, the human factor (mistakes in the design) plays a significant role. The author provides recommendations on this basis.

Thus, proposals for the determination of residual bearing capacity of damaged reinforced concrete bending elements of T-shaped profile in the available literature was not found.

It should also consider the strengthening of reinforced concrete bending elements, as the decision on the choice of a means of enhancing directly depends on the type of damage, the residual bearing capacity of the construction and its stress-strain state during operation.

After completing a preliminary analysis of methods for strengthening reinforced concrete constructions it can be summarized that we can enhance the constructions working on the bend both in a way of increasing cross-section and by the method of replacing a design scheme of work of constructions. Often the strengthening of reinforced concrete beams is performed by increasing the working reinforcement and device of clips. The increase of the bearing capacity of constructions is also possible to perform by simultaneous replacement of their design scheme and stress state. This type of strengthening is the most modern and widely studied at the present time and may exist without unloading constructions, and its effectiveness is determined only by the degree of pre-tension. Among the large number of methods for such strengthening are gaining of stretched reinforced concrete by the reinforced and the unreinforced polymer concrete [22], a layer of steel fiber reinforced concrete [32, 71...73, 105, 107], gaining of section of stretched reinforcement [37, 95], bonding with an epoxy adhesive a sheet of the reinforcement [66], increasing by acrylic polymer mortars [103], adhesion of composites on the basis of carbon and basalt fibers [54, 55, 65, 67, 119, 123, 125, 126,] etc.

The issues of strengthening based on steel fiber reinforced concrete were studied by B. A. Boyarchuk [22], G. V. Getun [32], O. P. Krichevsky [71], S. O. Krichevsky [72], A. N. Kulikov [73], O. P. Sunak [105], G. K. Khaidukov [107] and others. Their studies have shown that fiber slightly enhance the strength of concrete in compression. From these considerations follows the need to enter the steel fiber reinforced concrete only in the stretched area. In this case, the height of the layer of steel fiber reinforced concrete may vary depending on the strength, cracking resistance or deformability of the structure. This confirms the possibility of using steel fiber reinforced concrete to enhance the tensile area of bent constructions.

The research of E. O. Grinevitch [37] studied the strengthening of reinforced concrete bending elements by pre-compression with additional external reinforcement not along the entire length of the structure but at the site of action of significant bending moments. The destruction of these constructions occurred as a result of disintegration of compressed concrete, before achieving yield stress in additional reinforcement.

In the work of Ya. V. Rimar [95] there was performed a test of reinforced concrete beams before strengthening and applying of reinforcement under load in the stretched zone. The experiment showed that it is possible to obtain the same effect of strengthening when varying the reinforcement area of strengthening and the corresponding load. As a result, it is suggested to introduce a corrective coefficient of operating conditions of the reinforcement $\gamma sr = 0.5...1.0$.

M. I. Kisilier [66] carried out studies of the work of bent reinforced concrete items with reinforcement sheet glued with epoxy adhesive in the stretched zone. It is established that the adhesive strength depends on the concrete class. The experiments revealed three patterns of destruction of glued beams: separation of the protective layer of concrete in the stretched zone, offset by an adhesive connection, fluidity of the external stretched reinforcement.

In the work of M. Yu. Smolyaninov [103] it is proposed three schemes of strengthening of bent reinforced concrete beams with acrylic polymer mortar under the action of short-term static load: reinforcement of the bottom of the beam; the reinforcement of stretched zone to the axis line; in the form of holder. Analysis of the results shows that the scheme and the thickness of the layer of strengthening of reinforced concrete beams have a significant impact on their bearing capacity.

One of the innovative ways of strengthening of bent reinforced concrete elements is the gluing of rods, tapes and blades of composites (AFRP, GFRP, CFRP, respectively: Aramid-, Glass-, Carbon Fiber Reinforced Polymer). One of the first who started to study constructions strengthened with carbon fiber-reinforced plastic was Meier U [125].

The beams strengthened by him showed an increase in bearing capacity and stiffness twice.

Ritchie P. [126] tested sixteen reinforced concrete beams with minimum reinforcement of normal cross-section to examine the effectiveness of reinforcement based on glass and aramid fibers. Experiments have shown that the stiffness of such constructions increased by 17...99% and strength by 40...97% depending on the type, number and orientation of material of strengthening.

Studying of the effectiveness of anchoring using the segments of the tapes additionally glued in the support area on the bottom of the beams did M. E. Kaminska and R. Kotynia [123].The beams tested by them with low reinforcement ratio have been destroyed due to delamination of the tape, which happened very rapidly.

The study of strength of reinforced concrete beams with oblique bending and torsion strengthened by glued composite materials in the form of CFRP tapes did R. Al-Mahaidi, A. Hii [119].The peculiarity of their research was to capture the detachment of the external reinforcement before the moment of destruction of constructions by means of photogrammetric.

Among recent research in this field in our country we should note works [54, 55, 65, 67].

Under the leadership of V. G. Kvasha there were performed laboratory tests of bridge T-shaped beams strengthened by carbon fiber-reinforced plasticon TPIssue 56 models and two full-scale beams, which were tested for effects of single and polycyclic load. In [54] described the strengthening of the existing bridge over the river Prut in Ivano-Frankovsk region. The conducted static testing of the bridge before and after strengthening have given the opportunity to establish the actual nature of distribution of effort from temporary load between the beams, the effect of inclusion in collaboration with existing beams of the applied plate and check the bearing capacity of normal sections of strengthened constructions of the span. The author developed a method of calculating the strength of normal sections of reinforced concrete T-shaped bending items strengthened by glued CFRP tapes on the basis of nonlinear deformation model [55].

M. D. Klimpush in his work [65] presents a program of experimental studies of eight samples of reinforced concrete beams, which at the type of reinforcement model the beams of span structures on a standard project “Soyuzdorproekt. Issue 56” and “Soyuzdorproekt.Issue 56D[1]”, and beams of industrial buildings covering of type series PK-01-05[2], Issue 1.The samples were reinforced by welded frame with a working reinforcement 4Ø12 class A400. Transverse reinforcement of wire Ø5 mm class Bp-I.The compressed shelf of the beam is reinforced with 1Ø12 class A400 and welded grid with working reinforcement 4Ø5 Bp-I. There were developed models of stress-strain state and method of calculation of reinforced concrete beams strengthened by composite tapes for I and II groups of limiting states, substantiated by laboratory research and tested on real objects of experimental building. The work presents experimental data on strength, endurance, fracture toughness and deformability, which proved that in case of inclusion the strengthening tapes in the work of the beam on bending, bearing capacity increases of 1.3 ... 1.5 times, the width of the crack opening is reduced to 1.8 ... 1.9, deflections are reduced by 25 ... 30%, increase in endurance of 2.5 ... 3.0 times.

A. P. Kononchuk conducted a number of studies [67] of reinforced concrete beams without reinforcement and reinforced by composite materials, which allowed us to explore the resource of the compressed zone of concrete of beams, as evidenced by their simultaneous destruction by compressed and stretched parts of the section. The author has developed an adapted calculation of the required cross-sectional area of the external composite reinforcement of strengthening with the observance of the requirements of DBN V. 2.6.-98:2009[3] and DSTU B. V. 2.6-156:2010[4].

Among recent works on strengthening the shelves of T- shaped

[1]Translator's note: in Russian - Выпуск 56Д
[2]Translator's note: in Russian - типоваясерияПК-01-05
[3]Translator's note: in Russian - ДБНВ.2.6.-98:2009
[4]Translator's note: in Russian - ДСТУБ.В.2.6-156:2010

reinforced concrete bent items it should be noted works of Yu. L. Izotov [50], Z. R. Likhov [74].

In [50] Yu. L. Izotov discussed the results of experiments on bent and compressed out of center reinforced concrete elements with a significant strengthening of the compressed zone with reinforcement. The research shows that the norms of [12] overestimate the bearing capacity of cross-sections with significant reinforcement ratio of the compressed zone. The author attributes this to the formation of micro cracks in the concrete (including the shrinkage), and partial detaching of reinforcement with concrete in place of their contact.

Z. R. Likhov [74] designed reinforced concrete truss beams of T-section with the shelf in the stretched zone and the minimum width of the edge, at the upper side of which is pre-compressed high-strength reinforcement. Floor slabs for such beams rest on the cuts of the bottom shelf, which leads to a significant reduction in the structural height of the overlap (in this case - 31%) and the volume of the masonry of walls of the building (building with a single-span for one beam - 2.44 m^3). Reduction in cost for materials by these factors for one beam at 6 m step – 33%.

The lack of shelf in the compressed zone of the beam is compensated by a pre-compressed high-strength reinforcement, the maximum strain in which is equal and can be twice higher than in non-stressed reinforcement.

In [74] there was proposed a method of manufacturing of reinforced concrete T-section beams with a shelf in the stretched zone and with a pre-compressed reinforcement in the compression zone comprising two step concreting and measures ensuring the stability of the reinforcing bars with their prior compression.

Based on the above, interesting and relevant should be called a work on studying of the stress-strain state of the damaged reinforced concrete beams.The cross-section of the beams would be a T-section, since its effectiveness and advantages in comparison with a rectangular, were proved in this section of the thesis. The shelf should be considered as the least studied factor as the damage in the beam. This section presents a sufficient number of studies to determine the residual load-bearing capacity of constructions, but none of them considers the damage in the shelf of a T-shaped beam and does not make available methods for its determination. The difficulty in solving this problem lies in the fact that as a result of partial damage to the beam a front of destruction does not pass parallel to the axis of symmetry and a flat bend becomes oblique.

1.2. Calculation of reinforced concrete elements, operating at an oblique bend

Due to the scientific developments of M. A. Borisova [21], P. F. Vakhnenko [24], S. I. Glazer [33, 34] a method of calculating the strength of normal section of elements operated at oblique bending was sufficiently developed.The issue was developed in the works of A. A. Sapardinovich, A. V. Semko, Obiako Amaechi David, Yu. M. Rudenko [82, 96, 99, 101].

In the work of A. A. Sapardinovich [99] the method of calculating the strength of the elements operating on oblique bending was applied to I-beam constructions.

[82, 101] are about the study of crack resistance of oblique bending reinforced concrete elements of T-shaped and Γ-shaped cross section.

Yu. M.Rudenko [96] suggested a practical method of calculating compressible and bendable reinforced concrete items for complex types of deformations using isostatic curves: isobent and isostat. The accuracy of the method is confirmed by experiments conducted by author and other researchers.

A great contribution to the study of oblique bending reinforced concrete beams was made by the scientists of Poltava National Technical University named after Yuri Kondratyuk: O. V. Boyko [16], P. F. Vakhnenko [25...27], E. P. Voskoboynik [31], V. V. Dobryanskaya [40], K. H. Dolya [41...43], A. B. Khanin [81], A. M. Pavlikov[28, 84...87, 108], N. E. Rogoza [79], A. A. Kharchenko [69].

In the work of E. P. Voskoboynik [31] the program of the experiment involved testing two series of samples (B-1 and B-2) of reinforced concrete rectangular beams of rectangular section with defects of manufacture (inaccuracy of installation of reinforcement cages) and the damage (destruction of the protective layer of concrete and corrosion of working reinforcement) (Fig.1.4).

The results of the tests of prototypes with substantial manufacturing defects and corrosion damage (series B-2, Fig.1.4.), confirm their actual work on oblique bending, as evidenced by the deformation diagrams based on the results of strain gauge measurements (Fig. 1.5).

A. B. Nosach in his thesis [81] assessed the significance of the influence on the crack resistance of the inclined section of a number of factors and found that the most influential factor is the relative span of the cut. In the course of experimental and theoretical research, the author found that with increasing angle of inclination of the power plane β from 0° to 20° of absolute and relative efforts of the formation of inclined cracks decreases.

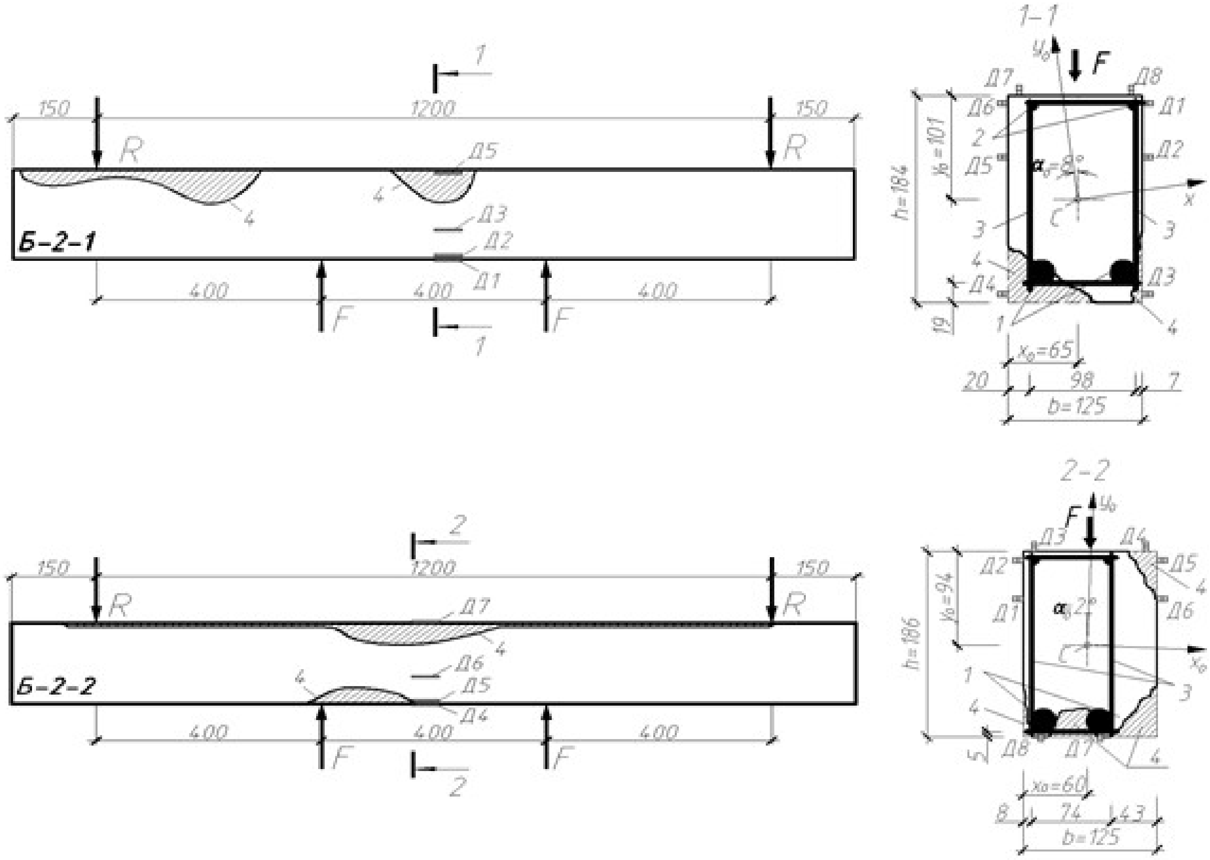


Fig. 1.4. The scheme of location of defects and damages and placing measuring devices in the testing of prototypes: 1 – longitudinal working reinforcement Ø22 A-III; 2 – longitudinal bars of the frame Ø10 A-I, 3 – the transverse bars Ø6 A-I, 4 – damage of the protective layer of concrete; D1 ... D8 – electric resistance strain gauges.

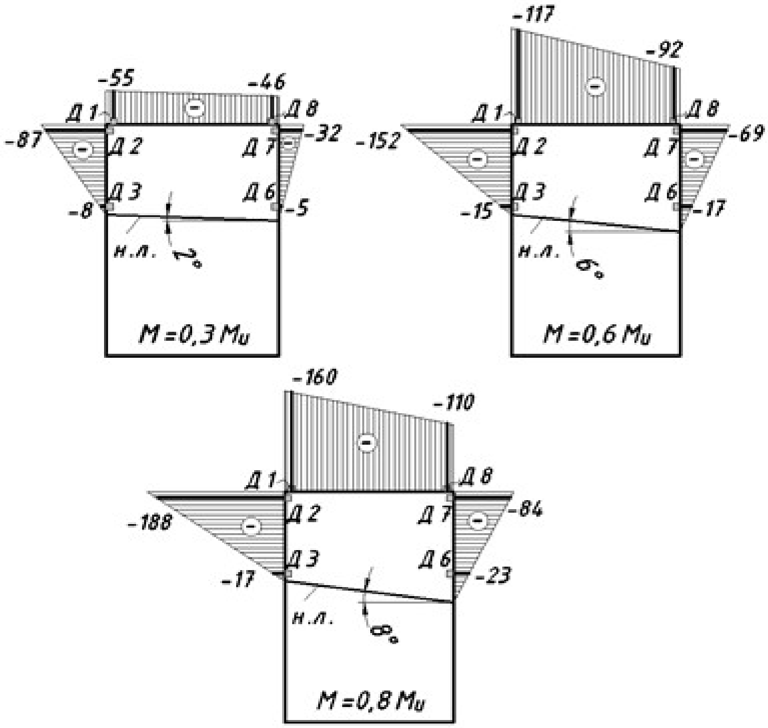


Fig. 15.Diagrams of deformations in the compressed zone of the concrete and the angle of the neutral line of the sample B-2-1 by the indications of electric resistance strain gauges.

When determining the efforts of the formation of inclined cracks one need to take into account the angle β of the power plane and changing the geometric characteristics of the cross section. It was determined that with increasing load the length of the projection of the threat of an inclined crack at the longitudinal axis of the element increases. To account for its change it is proposed to use the expression $c_w = c_0\sqrt{\frac{P}{P_u}}$. In the calculations it is proposed to take the length of the projection of the inclined crack not more than $2h_{0,eq}$. The imposed restrictions allow preventing the understatement of the efforts of the formation of inclined cracks and take into account the actual strain deformed state of the inclined section in the stage of formation of inclined cracks. To account for the uneven distribution of stresses in the transverse reinforcement along the length of the span of the cut when defining the width of opening of inclined cracks it is proposed to use the coefficient $\gamma_{sw} = 1 + \frac{c_w}{10s}$ which allows describing in detail the stress-strain state of the transverse reinforcement in the opening stages of inclined cracks.The method of evaluation of the crack resistance of the inclined section developed by A. B. Nosach allows calculating reinforced concrete elements under action of an oblique and flat bending, and takes into account the main factors affecting the crack resistance of the inclined section.

In the work of A. M. Pavlikov, O. V. Boiko [84] there were obtained expressions for determining the angle θ, allowing performing calculations without using iterative methods, and facilitating the method for determining the strength of reinforced concrete elements, operating at an oblique bending.

P. F. Vakhnenko and N. E. Rogoza [28] determined the main parameters of the stress-strain state in the reference sections of pre-stressed reinforced concrete T-shaped elements under oblique bending. In the future, on the basis of these studies, there was developed a method of determining the strength of such elements and directions for their optimal design.

A. M. Kharchenko [69] modeled the stress-strain state of reinforced concrete beams of T-shaped profile at the time of the destruction of the oblique bending. The proposed spatial model of the stress-strain state of oblique bent reinforced concrete beam of T-section based on nonlinear deformation model displays the actual operation of the item and allows to fully assessing its strength taking into account the characteristics of physical and mechanical properties of concrete, as a pseudo plastic material.

The works of N. I. Karpenko [52, 53], T.P. Chistova [111], N. N. Iachmeneva [116] study the deformability of items of T-section in bending and torsion.

From the conducted literature review it is clear that the work of the damaged bent reinforced concrete beams of the T-section in the process of operation has been studied little. The peculiarity of such constructions is that oblique bending in their cross-section is being implemented because of a violation (damage to the shelf) of symmetry of the section and changing the position of the principal axes of the already damaged section.

1.3. Conclusions for the Chapter and objectives of the research

The previously mentioned results of experimental and theoretical researches of work of the undamaged and damaged bending reinforced concrete elements of T-section allow drawing the following conclusions:

1. The current norms [68] do not include the possibility of determining the residual bearing capacity of damaged reinforced concrete beams ofT-section, although it could significantly reduce the costs of its strengthening, and the study of stress-strain state of such constructions would allow to analyze them in future work together with the design of strengthening.

2. In the process of operation the constructions of buildings and structures are exposed to factors that adversely affect their condition that causes the early wear of the latter and leads them in unserviceable condition. At the same time the damages which significantly reduce the bearing capacity of the item are added to the defects existing from the moment of manufacturing.

3. A sufficient amount of works disclose the research of damage of the edge of the T-shaped beams, as evidenced by the overview above. However, in the process of operation of such constructions the destruction of the compressed zone of the concrete of beam shelves occurs quite often. This section presents a number of studies on determination of residual bearing capacity of constructions, but none of them considers the damage in the shelf of T-beam and makes available methods for its determination.

4. The research of Poltava scientific school gives the possibility to calculate the oblique bent constructions with high confidence results that has been confirmed experimentally. However, this research was not used to study and calculate the strength of the already damaged T-shaped beams, in operation of which the front of destruction is not parallel to the axes of symmetry and flat bending becomes oblique.

The main objective of the work is to study the stress-strain state and work of reinforced concrete beams of T-section damaged during the operation, and having analyzed the influence of the main factors on the strength of such constructions, to develop a method for determining their residual bearing capacity.

To achieve this objective, the following tasks are set:

1. To develop the technique of experimental researches of reinforced concrete beams of T-section damaged during exploitation.

2. To conduct a full experiment and to set: the features of the action of damaged reinforced concrete beams of T-section under load; destruction properties; to analyze actual stress-strain state of such constructions with electrotensometry.

3. To perform a computer simulation of operation of damaged reinforced concrete beams of T-section with the non-linear action of concrete with the help of modern software complexes for the purpose of studying stress-strain state and determining the effect of various important factors on the residual bearing capacity of damaged items.

4. To develop a methodology for determining the residual bearing capacity of reinforced concrete beams of T-section on the basis of the main provisions of the regulations, as well as experimentally and theoretically obtained dependencies, and make recommendations on its use.

5. To carry out testing of the proposed methodology.

CHAPTER 2.
THE EXPERIMENT AND RESEARCH

2.1. The experiment on the study of stress-strained state of the damaged T-shaped beams.

As the analysis of literary sources has shown, the stress-strain state of the damaged bent elements of T-section prior to complete exhaustion of their bearing capacity remains poorly studied. Determination of the residual bearing capacity of such constructions would save a lot of money spent on the dismantling of constructions and their strengthening, which often has a significant margin of safety and is inappropriate. To solve these challenges, a series of tests using the theory of mathematical experimental design[119,120] was performed at the department of building constructions of Odessa state Academy of construction and architecture. This theory allows to set a theoretically justified minimum required number and composition of experiments to obtain in-depth information about qualitative and quantitative influence of the studied factors on the output parameters, both individually and their interaction, which cannot be achieved using traditional methods.

In a number of experimental and theoretical works performed in recent times there was shown the acceptability of methods of planning of experiment to study the behavior of reinforced concrete elements under load. The application of methods of experiment planning can improve its efficiency, reduce material costs, and identify the specific contribution of individual factors on the value of the latter functions.

The concept of using of mathematical methods of experimental design is known for more than 40 years. The works of its founder R. Fisher were developed by G. E. P.Box, I. Kiefer, and caused the emergence of new directions in applied mathematics, which is called experimental design.

When generalizing methods of experimental design, these positive qualities should be noted:

- the method allows to specifically create a random situation, in order to get rid of the need to stabilize the changing factors, as they are transformed into the random variables by bringing the experimental conditions in random;
- the method allows the experimenter to take into account the impact of the factors, which are usually neglected during the single-factor experiment;
- with the increase in the number of independent variables, the variance in the estimates of the regression coefficients is reduced;
- compared to single-factor experiments, multifactor allows reducing the radius of the studied sphere by the properties of the multidimensional space, resulting in dramatically increasing the efficiency of the experiment;
- the experimental design specifies a clear logic for all operations; this drastically reduces the amount of experimental studies.

Multivariate regression analysis is very sensitive with regard to violations of the initial assumptions on which the theory is based.Checking these experimental conditions is not always possible, so a decision on the extent to which they are executed is often taken intuitively.

Based on the analysis of a priori information from the literature taking into account the real possibilities of implementation as factors of variation accepted:

- the damaged part of the shelf expressed by the ratio (b_{eff1}/b_{eff2}), where $b_{eff1}$is the amount of damage, and $b_{eff2}$is the amount of overhang of the shelf;

- the depth of damage a_1, expressed through the ratio of the depth of damage of the shelf to the thickness of the shelf ($a_1/h_f^{/}$);

- the angle of damage β, expressed by the ratio of the angle of the damage to the angle of inclination of the shelf, equal to 90^0 (Fig.2.1).

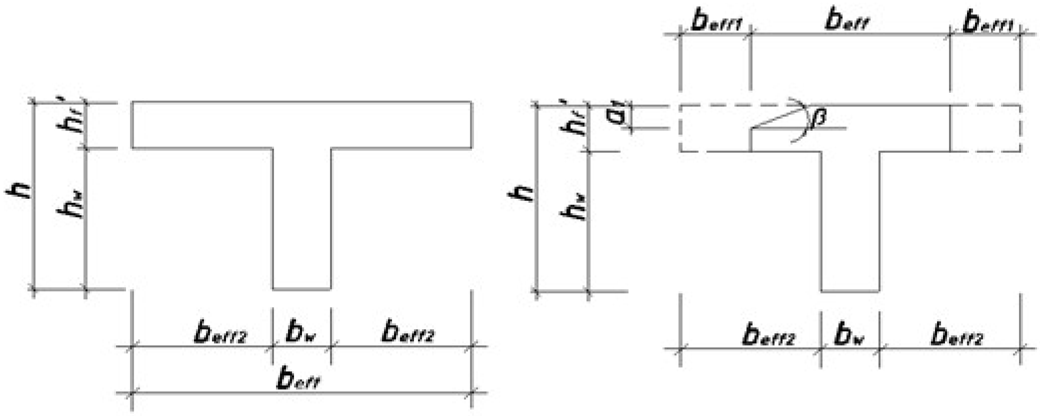


Fig. 2.1.The cross section of the undamaged beam and a beam with a damaged shelf.The levels and range of variation in these factors are summarized in table 2.1.

Table 2.1

Factors of variation for the three-factor model of experimentaldesign

The studied factors of Y series		Levels of variation			The interval of variation
Natural value	code	«-1»	«0»	«+1»	
the angle of damage β/90^0	X_1	00/900=0	22.50/900=0.25	450/900=0.5	0.25
the depth of damage $a_1/h_f^{/}$, mm	X_2	0/60=0	30/60=0.5	60/60=1	0.5
the damaged part of the shelf b_{eff1}/b_{eff2}, mm	X_3	0/165=0	82.5/165=0.5	165/165=1	0.5

Table 2.1 summarizes the full-scale values of the variable factors (X_1, X_2, and X3) and their levels and change intervals. The transition to normalized variables $-1 \le x_i \le +1$ is made according to the standard formula $x_i = (X_i - X_{i.0})/\Delta X_i$.

To determine the index of the strength of reinforced concrete beams there was conducted an experiment on 15-point D-optimal three-factor plan. The plan of the experiment is transformed in the direction of the main ($X_{i.0}$) and maximum levels ($X_{i.max}$) of the factors

X_2 and X_3, which allows a more detailed assessment of the effect of the depth of the damage and the size of the damaged part of the shelf-beams on samples analyzed criterion. Since in real conditions of operation of concrete beams with their expertise and technical condition it is quite difficult to perform accurate measurement of the angle of the damage (because of the rough chipped surface etc.), so the factor X_1 in terms of the experiment is expressed at extreme levels ($\beta=0^0$ and $\beta=45^0$) more in comparison with its median level ($\beta=22.5^0$).The experiment plan in the normalized variables and real values for variable factors is presented in table 2.2. [121].

2.2. The methodology of the research

2.2.1. The design of the experimental samples and the technology of their manufacturing. For experimental studies it was produced 15 experimental samples - T-section beams with injuries, as well as 45 concrete cubes and 45 concrete prisms to determine the strength and deformability of concrete characteristics of the concrete.

For the manufacture of experimental samples of beams, cubes and prisms a conventional heavy concrete with a design class C30/35 of prefabricated granite gravel fractions 5 to 20 mm, sand with a fineness modulus of 2.4 was used.

An ordinary Portland cement grade 400 without impurities [122] was used as a binder. In the process of concreting of beam samples in each experience with the same concrete mix there were produced 3 concrete cubes of 100×100×100 mm and 3 prisms of 100×100×400 mm, which were subsequently tested at the age of 90 days one day prior to the main trial in accordance with the requirements of the applicable regulations [123, 124]. The manufacture of all prototypes was performed on Kulindorovsky construction materials plant (20-th km of Starokievsky highway) in split steel casing.

Table 2.2

The matrix of the planned experiment

№	Label of the beam	The coded values of the factors			Natural values of the factors		
		X_1	X_2	X_3	The angle of damage $\beta/90^0$, (β, ^{0})	The depth of damage $a_1/h_f^/$, (a_1, mm)	The damaged part of the shelf b_{eff1}/b_{eff2}, (the value of the remaining shelf b_{eff}, mm)
1	B1	+1	+1	+1	0,5 (45^0)	1 (60)	1 (70)
2	B2	-1	+1	+1	0 (0^0)	1 (60)	1 (70)
3	B3	+1	+1	-1	0.5 (45^0)	1 (60)	0 (400)
4	B4	+1	+1	0	0.5 (45^0)	1 (60)	0.5 (235)
5	B5	-1	-1	+1	0 (0^0)	0 (0)	1 (70)
6	B6	-1	-1	-1	0 (0^0)	0 (0)	0 (400)
7	B7	0	0	0	0.25 ($22,5^0$)	0.5 (30)	0.5 (235)
8	B8	0	+1	+1	0.25 ($22,5^0$)	1 (60)	1 (70)
9	B9	+1	0	+1	0.5 (45^0)	0.5 (30)	1 (70)
10	B10	+1	0	0	0.5 (45^0)	0.5 (30)	0.5 (235)
11	B11	-1	0	0	0 (0^0)	0.5 (30)	0.5 (235)
12	B12	0	0	-1	0.25 ($22,5^0$)	0.5 (30)	0 (400)
13	B13	-1	0	-1	0 (0^0)	0.5 (30)	0 (400)
14	B14	0	+1	0	0.25 ($22,5^0$)	1 (60)	0.5 (235)
15	B15	-1	-1	0	0 (0^0)	0 (0)	0.5 (235)

Damages were only in the shelf of samples, the edge was left intact before testing.The length of the experimental samples was 2000 mm, the size of the shelves depending on the damage of the sample were $b_{eff}\times h_f^/$= (400; 235; 70) × (60; 30; 0) mm; edge width was the same for all 15 samples and ranged b_w = 70 mm; the height of beams h = 250 mm.

Reinforcement of the experimental samples of beams was made single Ø16 A500C (working reinforcement) and Ø6A240C (cross-section and mounting reinforcement) (Fig.2.2). The step of transverse rods was taken 100mm. Reinforcement of a shelf - 3Ø6 A240S longitudinally along the edges and in the middle of the shelf; laterally - Ø6A 240c with the same step as the transverse reinforcement of frame of the beam edge. Reinforcement of damaged parts of the shelves is missing for the purity of the experiment.

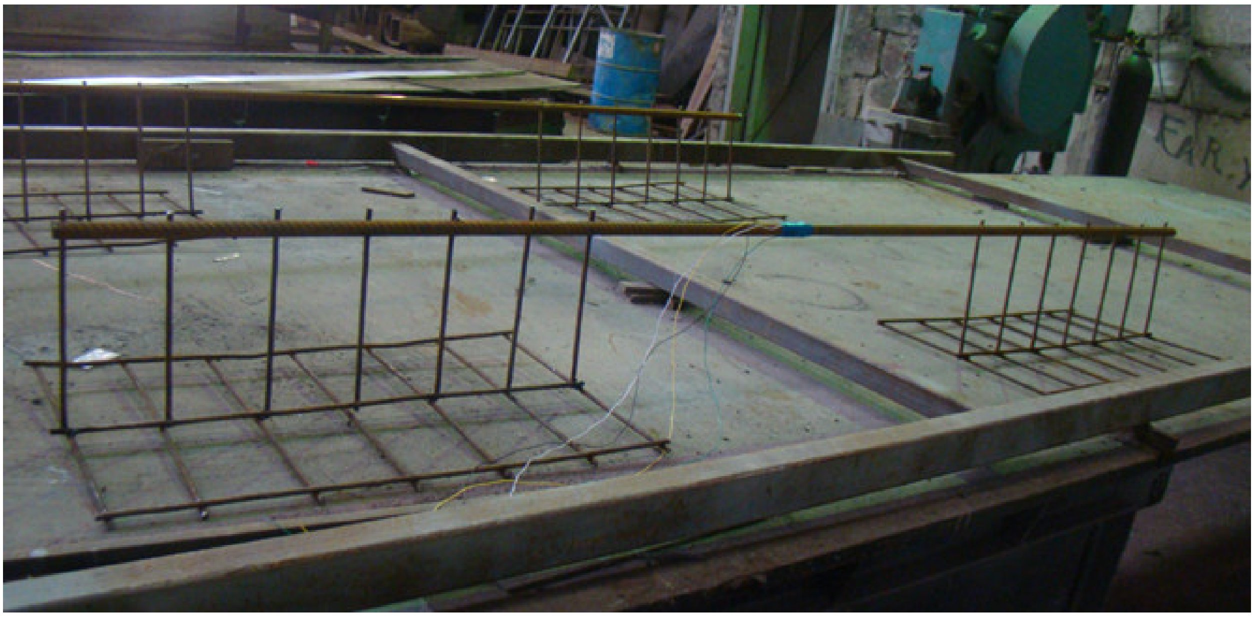


Fig.2.2. Reinforcement products for experimental samples of the beams.

In the supporting part of beams outside the span of the cut there were located mounting tabs (Ø6 A240C), that excluded their influence on the stress-strain state of a sloping section. The rods of the longitudinal and transverse reinforcement were connected in spatial frames with welding on industrial equipment (Fig. 2.3).

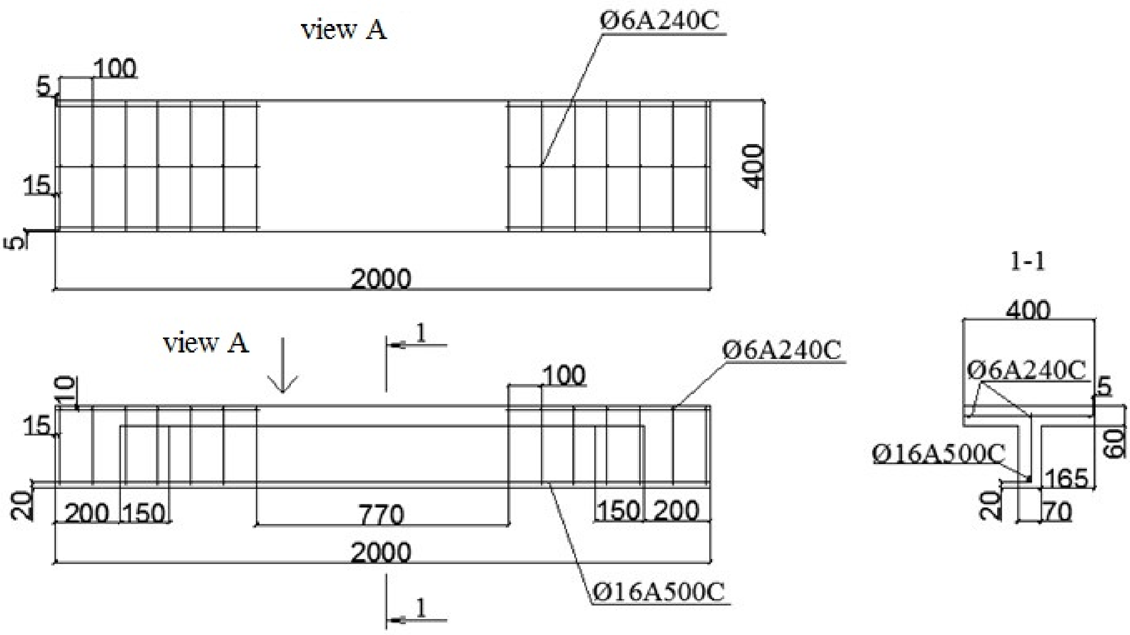


Fig.2.3. The reinforcement of the beams.

On the working reinforcement of the carcass (before the concreting of beams) two electric resistance strain gauges KF2P1 - 20 - 200A12 with a base, respectively, 20 mm and 200 ohms were pasted in advance.The wires for the further connection with fuzz of the recording device (in our case VNP-8[5]) were soldered to them.The resistance strain gauges were pasted with glue “Cyanofix” on surface of the reinforcing bar previously cleansed from rust and traces of a mortar. For a snug fit of the sensor to the reinforcing rod the flanges of the latter were sawed (Fig.2.4). The location of the resistance strain gauges was made symmetrical relative to the center of the bar at its maximum voltage. To avoid tearing of contact of the resistance strain gauge at concreting, their isolation was performed using the coil with isolation tape.

Fig. 2.4. Electric resistance strain gauge KF2P1 – 20 – 200A12[6] glued on the stripped surface and isolated.

Concreting of the samples was carried out in the individual formwork made directly at the place of concreting (Kulindorovskiy factory of building materials). The surface of the form was treated with special oil. One form was designed for two experimental samples (Fig. 2.5).

[5]Translator's note: in Russian – ВНП-8
[6]Translator's note: in Russian – КФ2П1 - 20 – 200А12

Fig. 2.5.Formwork for concreting experimental samples.

Fig. 2.6 shows the formwork for concreting of standard samples of cubes and prisms. Formwork was made of sheet steel and provides a reliable achievement of specified sizes. The surface of the formwork was lubricated with industrial oils before concreting.

Fig. 2.6.Formwork for the cubes lubricated with industrial oil.

Damage in the samples was artificially created with inserts of expanded polystyrene according to the experimental design in accordance with the type of damage (Fig. 2.7).

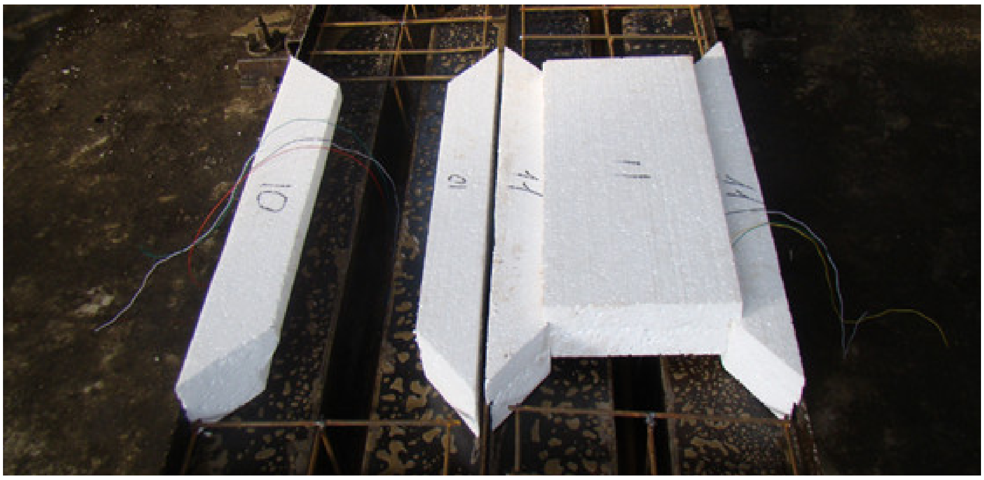


Fig. 2.7. Installing the expanded polystyrene inserts to create artificial beam damage.

For preparation and transportation of concrete mix a concrete mixer lorry was used (Fig.2.8), for that purpose in a mixing cylinder at the plant a dry concrete mixture was loaded, which for 5...8 min before arrival to the place of laying concrete mixture was mixed with the water pouring into the cylinder. The finished mixture was discharged by rotating the mixing cylinder in the opposite direction. A folding discharge tray allowed to do batch unloading. The concrete mixture delivered to the place of concreting was discharged to receiving hoppers (intermediate container), for subsequent transfer to the concrete blocks.

Fig. 2.8.Transportation of concrete mix to the place of unloading using concrete mixer lorry.

Before concreting the preparatory works were carried out:

- on the formwork – checking the major tick marks, geometric dimensions, verticality, absence of cracks, presence of inserts;
- on the reinforcement - quality of welds, correctness of installation, fixing reliability, providing a protective layer of concrete.

Designing location of reinforcement carcasses and grids was ensured by the correct installation of supporting devices: templates, retainers, and shims. To verify the compliance of the reinforcement to its strength characteristics three samples of the reinforcement were selected to test. The distance from the reinforcement to the nearest surface of the formwork was checked by the thickness of the protective layer of concrete of the concreting beams.

The reinforcement for a firm grip with fresh concrete mixture was purified from dirt, loose rust and adhere pieces of solution with a sandblasted machine and wire brushes.

Since experimental samples – beams are small in terms of the design; the laying of concrete mixture was carried out immediately at full height without interruption to prevent the device of working joints.

The concrete mix was placed in a concreted design in horizontal layers of approximately uniform thickness and without gaps along the length and with consistent direction of putting to one side in all layers (Fig. 2.9).

Fig. 2.9.Submission of the concrete in the concreting block.

Compacting of concrete mix in forms of samples of cubes and prisms of samples was performed on a vibrating table and samples of beams by deep vibrators. When compacting the concrete mixture, the resting of vibrators on reinforcement, inserts and other elements of formwork was not allowed (Fig. 2.10).

The duration of the vibration provided a sufficient compacting, the main features of which are:

- the termination of subsidence of a laid concrete mix;
- the appearance of the laitance on its surface;
- the termination of discharge of air bubbles on the surface.

Fig. 2.10.Compacting of concrete mix with a deep vibrator.

In the operation process of concrete works the condition of the formwork and embedded parts was constantly monitored.

2 samples of experimental beams were been concreted at the same time. At each concreting from the same batch there were made three cubes with edge of 100 mm and three prisms of size 100×100×400 mm. The top layer of the concrete was levelled and smoothed with a spatula (Fig. 2.11).

During the first 14 days after concreting experimental samples – beams, control cubes and prisms were covered with burlap; its surface was moistened with water and covered with a polyethylene to prevent excessive moisture loss. After 14 days the demoulding of samples, cubes and prisms was made (Fig. 2.12).

Fig.2.11. Alignment of experimental samples.

Fig. 2.12.The dismantling of experimental samples – beams.

2.2.2. Methods for determining the strength and deformability of concrete and reinforcement

For the manufacturing of experimental samples - beams, cubes and prisms a conventional factory-made heavy concrete with a design class C30/35 on granite gravel of fractions 5 to 20 mm, sand with a fineness modulus of 2.4 was used. An ordinary Portland cement 400 without additives was used as a binder. In the process of concreting of experimental samples - beams in each experience there were made 3 concrete cubes of 100×100×100 mm and 3 prisms of 100×100×400 mm of the same concrete mix, which were subsequently tested at the age of 90 days and one day prior to the main trial in accordance with the requirements of the applicable Regulations [123, 124].

For the expected value of the breaking load of the concrete cubes there was accepted the maximum force achieved during the test. The limit of strength of concrete in compression $f_{ck,cube}$(kgf/cm2) was calculated by the formula (2.1)

$$f_{ck,cube} = {}^{F_{max}}/_{A} \qquad (2.1)$$

where$F_{max}$is breaking load, kgf;
A is the average cross-sectional area of the sample cube, cm.

The determination of characteristics of concrete $f_{ck,prism}$and E_{cm} in accordance with [125] was carried out before testing of each batch of sample beams.In determining prism strength of concrete in compression $f_{ck,prism}$ according to GOST 24452 80 "Concretes. Methods of determining prism strength, modulus of elasticity and Poisson's ratio“ [126] prisms of size of 100×100×400 mm were brought to a destruction with step-increasing load of the press P-125 No. 1247 GOST 8905-67 (Armavir, year of manufacture – 1974), the scale interval was 125kg. The value of the degree of loading was equal to (0.05 ... 0.10) of the expected failure load and assumed to be constant throughout the tests. The application of the load was centered by the preliminary control compression. For this purpose after installation of the prism and centering on the geometric axis it was subjected to compression with the force of 0.1 F_{max}. The estimated value of the breaking load was determined by the results of the test of the cubes by the formula (2.2)

$$F_{max} = 0.75 f_{ck,cube} \cdot A, \tag{2.2}$$

where A is the average cross-sectional area of the prism, cm^2;
$f_{ck,cube}$ is the cube strength of concrete in compression, kg/cm^2;
0.75 is the average transition ratio from cube strength to prismatic.

During the test, the longitudinal and transverse deformations of concrete were measured by using indicating gages with scale interval of u1=0.001 mm and a base of measuring 200 mm, which were located on two opposite sides and fastened to the body of the prism with special seat frameworks by means of anchor screws. To control the accuracy of the measurements carried out, there were also used electric resistance strain gauges on a paper basis with a base of 50 mm, glued along the central axes of the side faces of the prism. The deformations were fixed at the beginning and end of each load step. The degree of loading was kept 4 ... 5 minutes.

The initial modulus of elasticity of concrete E_{cm} was determined with a load of 30% of the destructing, according to the formula (2.3)

$$E_{cm} = \frac{\sigma_1}{\varepsilon_{1y}}, \tag{2.3}$$

where $\sigma_1 = F_1 / A_c$ is the increment of stress from conventional zero to the level of the external load equal to 30% of the destructive;
F_1 is the appropriate increase of the external load;
ε_{1y} is the increment of an elastic-instantaneous relative longitudinal strain of the sample corresponding to the load level and the load measured at the beginning of each stage of its application (determined by linear interpolation).

For all experimental samples reinforcement of the same batch was used. To determine the physical and mechanical properties of reinforcing steel according to [127] there has been selected 3 samples of reinforcement of each class of 500 mm length.

The area of the reinforcing bar was determined by the formula (2.4)

$$A = \frac{ml}{\gamma}, \tag{2.4}$$

where$m$ is the weight of bar in grams;

$\gamma = 7.9$ g/cm^{3}is the specific gravity of steel;

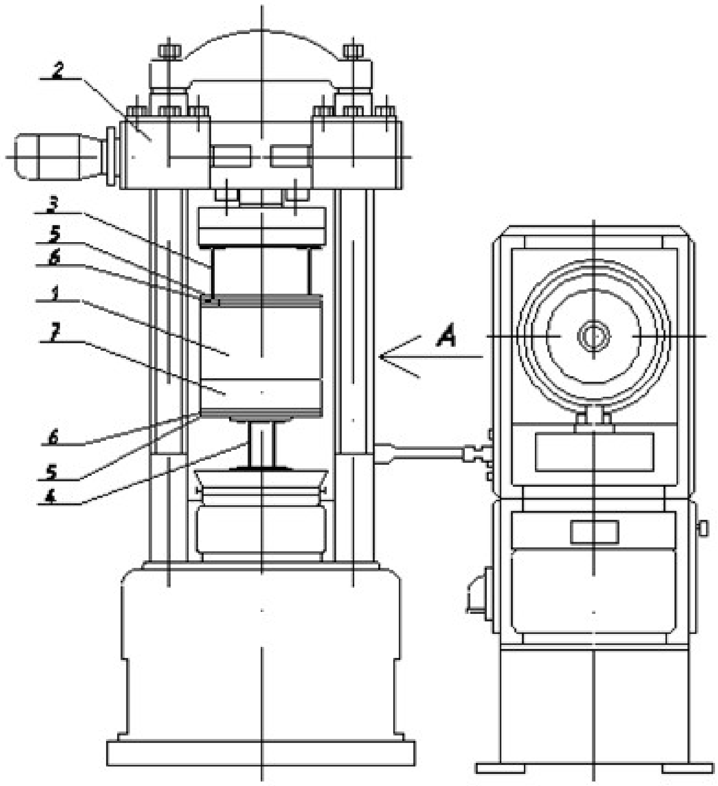
l is the length of the bar, cm.

According to [127] the test of each bar was carried out by stepwise-increasing workload with the shutter speed at each step for the samplingsof instruments.The tests were carried out on a tensile testing machine UIM-50[7] (scale of measurement 100...500 kN). During testing the following characteristics were determined: yield strength f_{yd}; temporary tensile strength f_u; modulus of elasticity of reinforcement E_s, elongation δ.

2.2.3. The methods of testing of experimental samples – beams

Testing of the T-shaped beams was carried out on hydraulic press P-125 No. 1247 GOST 8905-67 (Armavir, 1974 year of issue) with scale interval of 125 kgf. The load on the beam was transmitted through two traverses to distribute the load across the joints (Fig.2.13). The upper and lower traverses consisted of two I-beams No. 16 with welded top and bottom plates of 400×800×10 mm for the upper traverse and 200×2000×10 mm for the lower traverse (Fig.2.14). The joints transmitted the load to the beam through metal plates of 200×400×10 mm for areas near the support and of 100×400×10 mm in the locations of application of concentrated load. One joint was welded to the plate and was considered to be stationary, and the second one freely rested on it (moving joint). For the convenience of fixing of deflections of beams and to avoid bearing the lower traverse during the testing, it was decided to raise the beam above the support joints at the height of 150 mm by means of putting under its support the inserts of heavy concrete of 400 × 200 × 150 mm.

[7]Translator's note: in Russian - УИМ-50



View A

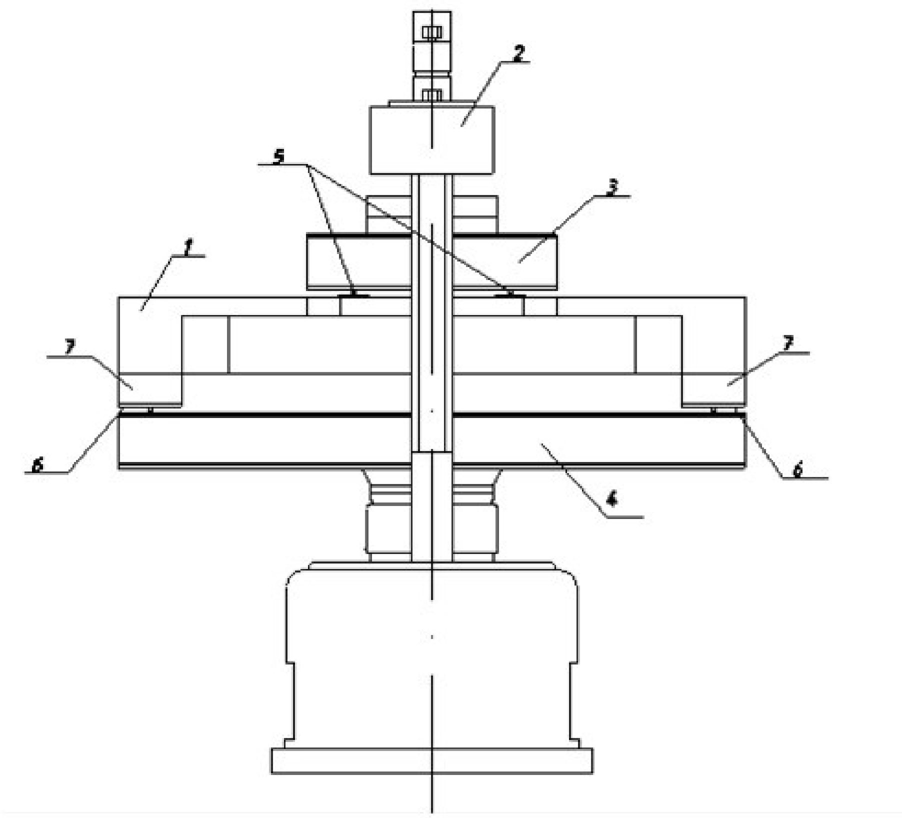


Fig. 2.13. Machine for testing of the damaged T-shaped beams:
1 – the experimental sample; 2 – hydraulic press P-125; 3 – upper traverse; 4 – bottom traverse; 5 – joints; 6 – distribution plates; 7 – the insert made of heavy concrete.

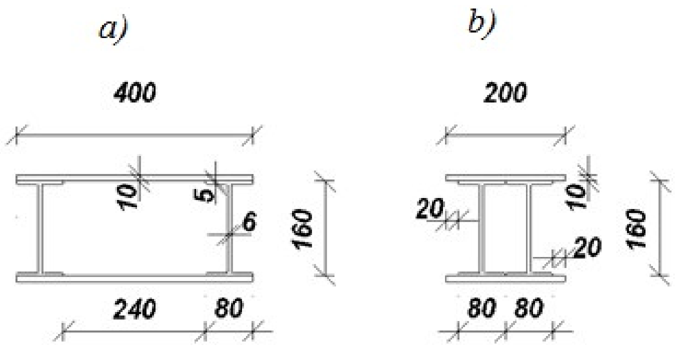


Fig. 2.14. Distributive traverses: a – top; b – bottom.

General view of the experimental samples in the series (depending on the remaining width of the shelf) is shown in Fig.2.15 and table 2.3.

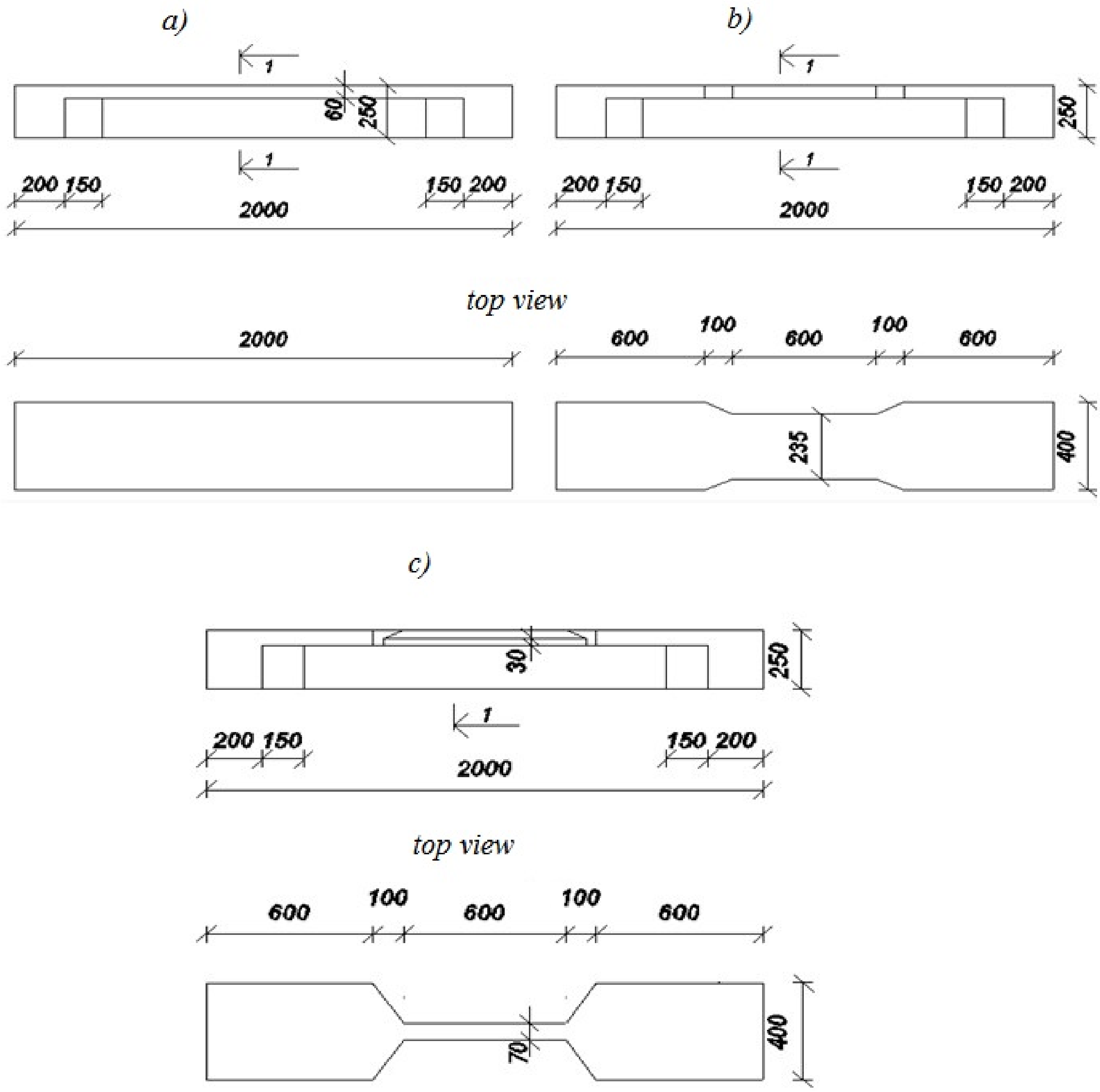
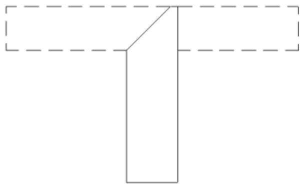
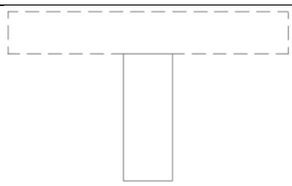
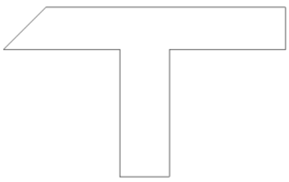
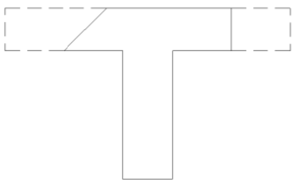
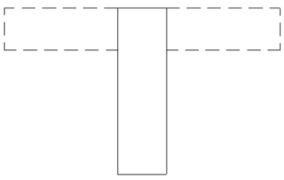
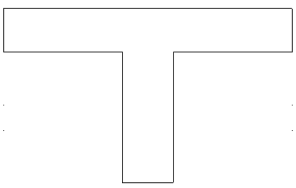
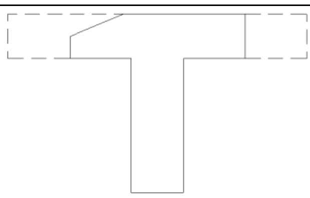
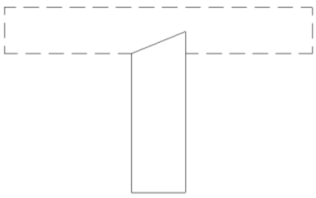
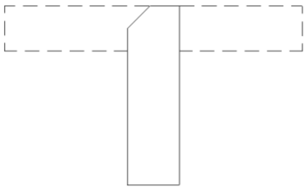
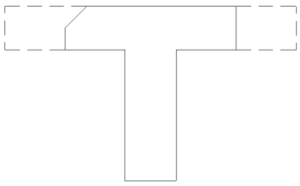
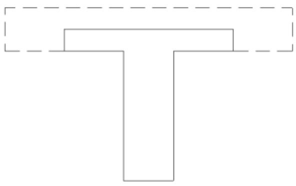
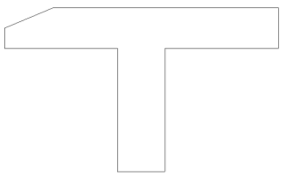
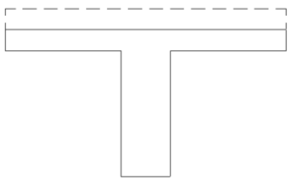
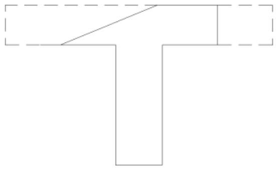
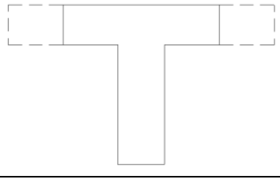


Fig. 2.15. General view of the samples on the series of beams (depending on the width remaining after damage to the shelf): a – b_{eff} = 400 mm, samples B3,6,12,13; b – b_{eff} = 235 mm, samples B4,7,10,11,14,15; c – b_{eff} = 70mm, samples B1,2,5,8,9.

Table 2.3

Features of the section 1-1 of experimental samples – beams

Label	Sketch	The angle of damage $\beta/90^0$, (β,^{0})	The depth of damage $a_1/h_f^{/}$, (a_1, mm)	The damaged part of the shelf b_{eff1}/b_{eff2}, (the value of the remaining shelf b_{eff}, mm)
1	2	3	4	5
B1		0.5 (45^0)	1 (60)	1 (70)
B2		0 (0^0)	1 (60)	1 (70)
B3		0.5 (45^0)	1 (60)	0 (400)
B4		0.5 (45^0)	1 (60)	0.5 (235)
B5		0 (0^0)	0 (0)	1 (70)
B6		0 (0^0)	0 (0)	0 (400)
B7		0.25 ($22,5^0$)	0.5 (30)	0.5 (235)

B8		0.25 (22.5^0)	1 (60)	1 (70)
B9		0.5 (45^0)	0.5 (30)	1 (70)
B10		0.5 (45^0)	0.5 (30)	0.5 (235)
B11		0 (0^0)	0.5 (30)	0.5 (235)
B12		0.25 ($22,5^0$)	0.5 (30)	0 (400)
B13		0 (0^0)	0.5 (30)	0 (400)
B14		0.25 ($22,5^0$)	1 (60)	0.5 (235)
B15		0 (0^0)	0 (0)	0.5 (235)

In the process of testing the value of deflection was fixed with indicating gages according to GOST 577 with a scale interval of 0.01 mm in accordance with paragraph 6.4 [123], located on either side of the beams on the lower edges of the samples; both transverse and longitudinal deformations of concrete and reinforcement using electric resistance strain gauges on the base of paper with a base of 50 mm for concrete and 20mm for the reinforcement and the resistance of 200 Ohms (Fig.2.16, 2.17).

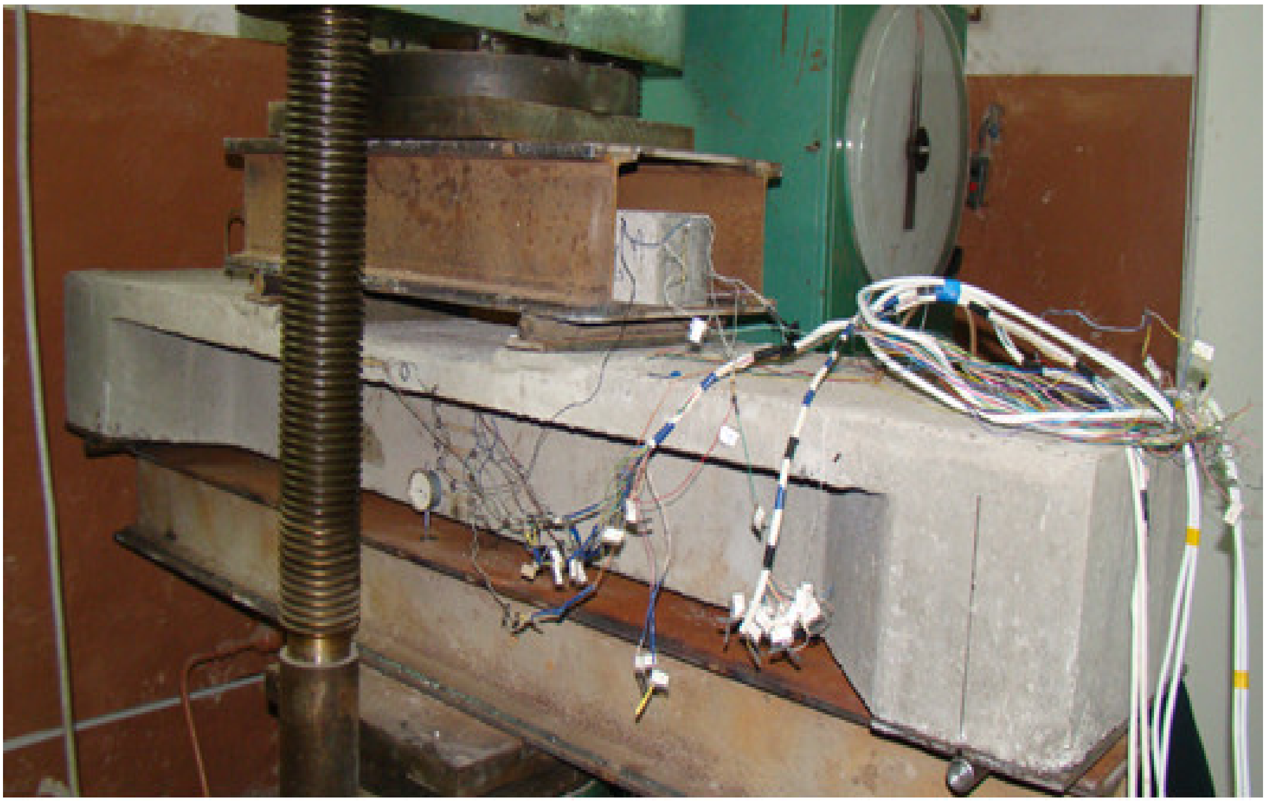


Fig. 2.16. The appearance of sample prepared for the testing.

Placement of electric resistance strain gauges on each sample was slightly different from one another, due to differences in the nature of the damage.

`Prior to gluing the resistance strain gauges, a preparation of places of measurement was carried out. The preparation of places included cleaning and leveling of the surface, priming and applying alignment marks.

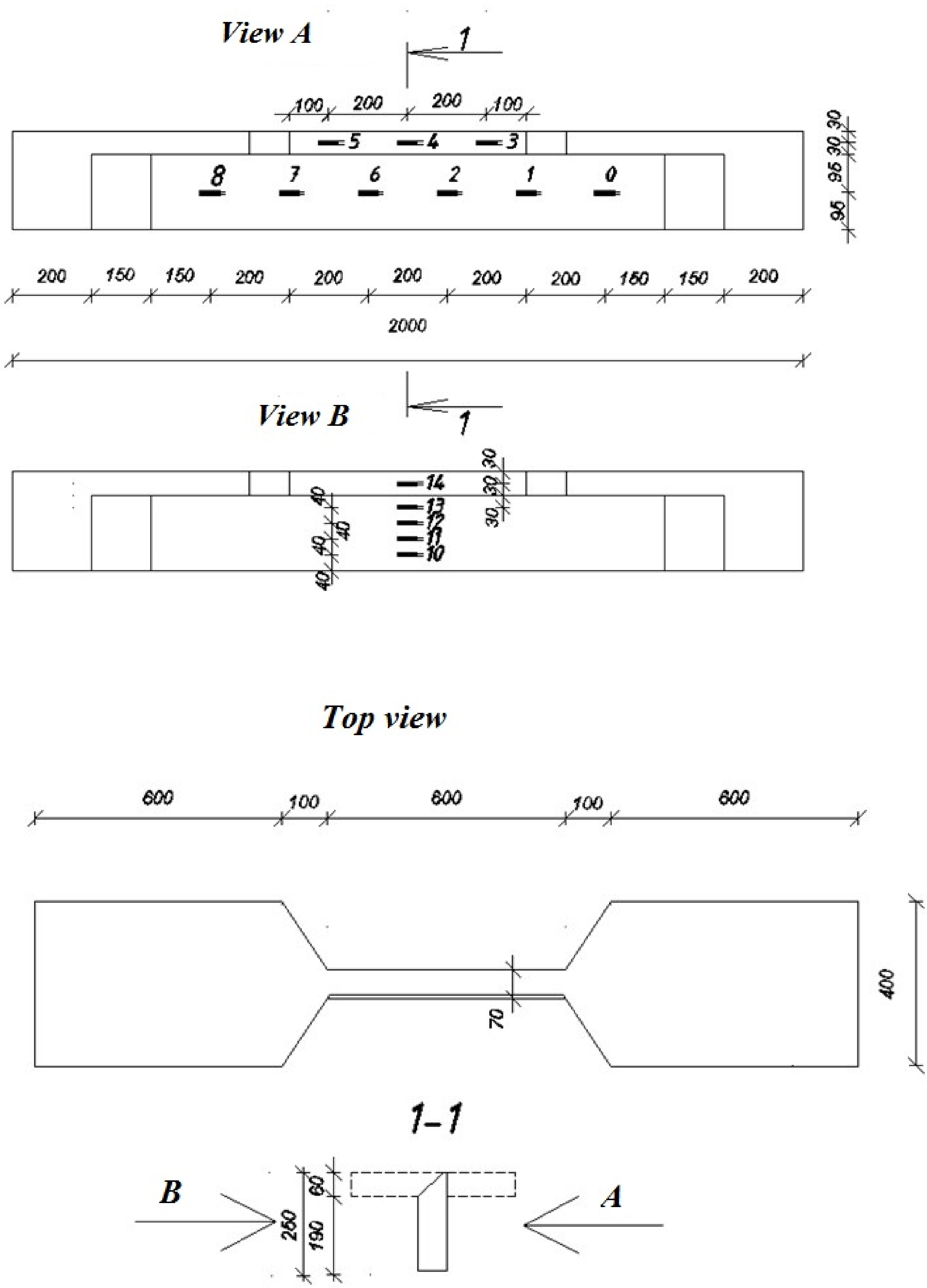


Fig. 2.17. The scheme of arrangement of electric resistance strain gauges on a sample B1.

First the concrete surface was cleaned from dirt, then using a manual abrasive tool was aligned and polished to cleanness, then the surface were smoothed out with a sanding No. 8.After cleaning the contact points were primed in order to form an intermediate adhesive layer, which increases the adhesion in subsequent gluing of electric resistance strain gauges.

Priming was performed with glue BF2 on a surface previously degreased with acetone.Immediately before gluing the electric resistance strain gauges were checked and sorted by assembling groups. All electric resistance gauges belonged to the same production batch. The range of resistance values provided the balancing of the system. Gluing of electric resistance strain gauges was carried out with a glue “Cyanofix”, which belongs to the cyanoacrylate group. Since the resistance strain gauges had a paper base they were previously primed with the glue BF2 with further polymerization. After applying Cyanofix the resistance strain gauges were placed on a demarcated place and kept under pressure for a minute. Further drying was carried out for 6 hours. Simultaneously with the gluing of the resistance strain gages, mounting pads made of isolating material with conductors for soldering the resistance strain gauges and switching wires were installed and glued.Mounting pads prevent separation of the resistance strain gauges during installation and eliminate the possibility of short circuits during measurement. The resistance strain gauges that were defective when appliedwas replaced by new, followed by verification of their reliability.

The measurement was started with checking the stability of the measuring path, which was achieved by repeated (3...10) reading of the initial (zero) samples in each working resistance strain gauge.

The fixing of the indications of the resistance strain gauges was carried out using a multi-channel strain gauge measuring system for static tests VNP8 (Fig. 2.18). For the successful observation of the measurement the indicators of electric resistance strain gauges were processed during the testing, and the variations of indicators did not exceed the bounds of unreliability.

The software consists of 3 main modules that provide various functions, and the monitor controlling the operation of the program. The software gives the opportunity [128]:

- to change the indications of the counter of the number of measuring channel with a frequency of (5 ±2) Hz;

- to change the indications of the counter of the number of measuring channel once;

- to remember the starting and ending number of measuring channels;

- to measure the readings of the selected channel once;

- to measure and record the status of the given array of channels automatically;

- to select the connection type of strain gauges.

Fig. 2.18.System VNP 8.

The data are presented in tabular form. The table is stored in the format *.xls (MS EXCEL) or in a different format, combined with MS EXCEL.

The load was transferred by stages with exposure at each stage. Before and after the exposure the readings on the devices were recorded at each stage of loading. The journal of observations recorded points of cracking, their disclosure, the nature of the sample destruction, photographic images were made. During the testing beams on the samples recorded the formation and further development of normal and inclined cracks using a marker or chalk at each stage of loading. Near the cracks the load value corresponding to the time of its formation was indicated. Separately in the journal of observations the deflections of beams by readings of dial indicators were recorded with further averaging (Fig.2.19).

Fig. 2.19. Beam during the test – fixing the resulting cracks and the readings on the indicators.

The inability of the experimental sample to perceive the growing workload was taken as the criterion of destruction.

2.3. Conclusions for the Chapter

1. The program of experimental studies adopted in the work allowed obtaining statistically reliable data on the impact of the three damage parameters on the stress-strain state and residual bearing capacity of damaged reinforced concrete beams of T-section.

2. The materials used in the manufacture of experimental samples of beams and the technology of their production meets current building regulations.

3. The methodology of conducting experimental studies that was used in the work (schemes of the layout of measuring devices and choosing of measuring equipment) has allowed to obtain reliable and sufficient information on the work of the damaged T-shaped beams and to characterize their stress-strain state, which will allow to develop an improved methodology for calculating the strength of such damaged constructions.

CHAPTER 3.

ANALYSIS OF THE RESULTS OF EXPERIMENTAL RESEARCH

3.1. The results of the tests of experimental samples

3.1.1. Cubes, prisms, reinforcing bars. To determine the strength and deformability of concrete, the tests of concrete cubes and prisms were carried out. Cube after the destruction was in the form of two truncated pyramids of closed lower base (Fig. 3.1), which corresponds to the standard nature of the destruction. The strength of the standard sample - cube was determined as the average of three samples of the series. The lowest result was no different from the next by more than 15%.

Fig. 3.1. The nature of the destruction of experimental samples - cubes.

The testing of experimental samples of concrete prisms (Fig. 3.2) allowed establishing prism compressive strength.

Counting was performed at the beginning and at the end of each stage of loading, the results entered in the test log (Table 3.1). Using the form of this table, the tension σ, relative longitudinal deformation ε_c and the initial modulus of elasticity of concrete E_{cm} were determined for each stage.

Fig. 3.2.The nature of the destruction of experimental samples - prisms.

The test results of cube and prism strength of concrete by series are shown in tables 3.1 and 3.2.

Table 3.1

Determination of deformation properties of the concrete

Number of loading stage	Load F, kgf	A_c, cm^2	Strain$\sigma=F/A$, kgf/cm^2	IndicatorNo.1		Indicator No.2		Average$\varepsilon_c\cdot10^{-4}=(\varepsilon_1+\varepsilon_2)/2$	$E_{cm}=\sigma/\varepsilon$, kgf/cm^2	$E_{cm}=\sigma/\varepsilon$, GPa
				u_1=0,001мм, l_1=200мм						
				count 1	$\varepsilon_1\cdot10^{-4}=n_1u_1/l_1$	count 2	$\varepsilon_2\cdot10^{-4}=n_2u_2/l_2$			
1	2	3	4	5	6	7	8	9	10	11
P 1/2										
1	0	100	0	0	0	0	0	0	–	
2	3000		30	40	2	0	0	1	300000	
3	6000		60	68	3.4	12	0.6	2	300000	
4	9000		90	94	4.7	32	1.6	3.15	285714.3	29.7
5	12000		120	123	6.15	58	2.9	4.525	265193.4	
6	15000		150	152	7.6	86	4.3	5.95	252100.8	
7	18000		180	180	9	120	6	7.5	240000	
8	21000		210	213	10.65	165	8.25	9.45	222222.2	
9	24000		240	253	12.65	242	12.1	12.375	193939.4	
10	25500		255	317	15.85	326	16.3	16.075	158631.4	

P 3/4										
1	0	100	0	0	0	0	0	0	–	
2	3000		30	8	0.4	18	0.9	0.65	461538.5	
3	6000		60	24	1.2	35	1.75	1.475	406779.7	
4	9000		90	40	2	56	2.8	2.4	375000	
5	12000		120	67	3.35	78	3.9	3.625	331034.5	
6	15000		150	73	3.65	100	5	4.325	346820.8	
7	18000		180	91	4.55	123	6.15	5.35	336448.6	36.8
8	21000		210	111	5.55	150	7.5	6.525	321839.1	
9	24000		240	131	6.55	178	8.9	7.725	310679.6	
10	27000		270	152	7.6	215	10.75	9.175	294277.9	
11	30000		300	205	10.25	286	14.3	12.275	244399.2	
12	31250		312.5	220	11	300	15	13	240384.6	
P 5/7										
0	0	100	0	0	0	0	0	0	–	
1	3000		30	15	0.75	13	0.65	0.7	428571.4	
2	6000		60	45	2.25	34	1.7	1.975	303797.5	
3	9000		90	73	3.65	55	2.75	3.2	281250	
4	12000		120	98	4.9	80	4	4.45	269662.9	
5	15000		150	123	6.15	108	5.4	5.775	259740.3	28.4
6	18000		180	148	7.4	140	7	7.2	250000	
7	21000		210	170	8.5	173	8.65	8.575	244898	
8	24000		240	195	9.75	215	10.75	10.25	234146.3	
9	27000		270	216	10.8	263	13.15	11.975	225469.7	
10	28250		282.5	221	11.05	305	15.25	13.15	214828.9	
P 6/15										
1	0	100	0	0	0	0	0	0	–	
2	3000		30	26	1.3	6	0.3	0.8	375000	
3	6000		60	56	2.8	20	1	1.9	315789.5	
4	9000		90	85	4.25	35	1.75	3	300000	
5	12000		120	114	5.7	54	2.7	4.2	285714.3	
6	15000		150	141	7.05	75	3.75	5.4	277777.8	30.0
7	18000		180	171	8.55	98	4.9	6.725	267658	
8	21000		210	205	10.25	128	6.4	8.325	252252.3	
9	24000		240	244	12.2	159	7.95	10.075	238213.4	
10	27000		270	298	14.9	204	10.2	12.55	215139.4	
11	30000		300	375	18.75	278	13.9	16.325	183767.2	
P 8/9										
1	0	100	0	0	0	0	0	0	–	
2	3000		30	22	1.1	18	0.9	1	300000	
3	6000		60	49	2.45	45	2.25	2.35	255319.2	
4	9000		90	78	3.9	80	4	3.95	227848.1	
5	12000		120	102	5.1	110	5.5	5.3	226415.1	

6	15000		150	130	6.5	150	7.5	7	214285.7	23.4
7	18000		180	158	7.9	190	9.5	8.7	206896.5	
8	21000		210	190	9.5	248	12.4	10.95	191780.8	
9	24000		240	234	11.7	325	16.25	13.975	171735.2	
10	26750		267.5	315	15.75	410	20.5	18.125	147586.2	
P 12										
1	0	100	0	0	0	0	0	0	–	
2	3000		30	30	1.5	5	0.25	0.875	342857.1	
3	6000		60	60	3	20	1	2	300000	
4	9000		90	85	4.25	36	1.8	3.025	297520.7	
5	12000		120	105	5.25	55	2.75	4	300000	
6	15000		150	151	7.55	178	8.9	8.225	182370.8	30.8
7	18000		180	208	10.4	110	5.5	7.95	226415.1	
8	21000		210	254	12.7	139	6.95	9.825	213740.5	
9	24000		240	332	16.6	186	9.3	12.95	185328.2	
10	25500		255	430	21.5	240	12	16.75	152238.8	
P 13/14										
1	0	100	0	0	0	0	0	0	–	
2	3000		30	30	1.5	11	0.55	1.025	292682.9	
3	6000		60	64	3.2	35	1.75	2.475	242424.2	
4	9000		90	88	4.4	64	3.2	3.8	236842.1	
5	12000		120	111	5.55	98	4.9	5.225	229665.1	
6	15000		150	141	7.05	132	6.6	6.825	219780.2	29.9
7	18000		180	175	8.75	172	8.6	8.675	207492.8	
8	21000		210	224	11.2	244	12.2	11.7	179487.2	
9	24000		240	274	13.7	348	17.4	15.55	154340.8	
10	24750		247.5	302	15.1	440	22	18.55	133423.2	
P 10/11										
1	0	100	0	0	0	0	0	0	–	
2	3000		30	20	1	12	0.6	0.8	375000	
3	6000		60	42	2.1	28	1.4	1.75	342857.1	
4	9000		90	65	3.25	49.2	2.46	2.855	315236.4	
5	12000		120	90	4.5	68	3.4	3.95	303797.5	
6	15000		150	117	5.85	88	4.4	5.125	292682.9	
7	18000		180	145	7.25	107	5.35	6.3	285714.3	24.0
8	21000		210	174	8.7	129	6.45	7.575	277227.7	
9	24000		240	206	10.3	153	7.65	8.975	267409.5	
10	27000		270	246	12.3	182	9.1	10.7	252336.5	
11	30000		300	290	14.5	210	10.5	12.5	240000	
12	33000		330	354	17.7	244	12,2	14.95	220735.8	
13	35000		350	417	20.85	276	13.8	17.325	202020.2	
14	35900		359	480	24	380	19	21.5	166976.7	

Table 3.2

Mechanical properties of concrete samples by series

Series of the beams	The average strength of concrete, MPa						The modulus of elasticity of concrete, MPa	The limiting relative deformation of compression
	$f_{ck,cube}$, (MPa)	the coefficient of variation v_c *, %	$f_{cm,cube}$ **, (MPa)	$f_{ck,prism}$, (MPa)	$f_{cd}=f_{ck,prism}/\gamma_c$, (MPa)	$f_{ck,prism}$ ***, (MPa)	E_{cm}, (GPa)	$\varepsilon_{c1,ck}10^{-4}$
B1/2	37	3.8	39.5	25.5	19.6	28.9	29.7	16.1
B3/4	40.1	1.8	41.3	31.25	24.0	30.1	36.8	13
B5/7	36.8	3.7	39.1	28.25	21.7	28.6	28.4	13,2
B6/15	37.5	11.3	46.0	30.0	23.1	33.3	30.0	16.3
B8/9	37.1	5.0	40.4	26.75	20.6	29.5	23.4	18.1
B10/11	39.1	12.2	48.9	36.0	27.7	35.3	30.8	21.5
B12	31.5	1.4	32.2	25.5	19.6	23.8	29.9	16.8
B13/14	28.7	2.0	29.7	24.75	19.0	22.0	24.0	18.5
$\overline{z}$	35.98			28.5			29.13	
S_z	3.87			3.80			4.19	
$v_c = \frac{s_z}{\overline{z}}$, %	10.8			13.3			14.4	

*) – $v_c = \frac{s_z}{\overline{z}}$ is the coefficient of variation

where $s_z = \left[\frac{\sum_{i=1}^{N} \left(z_i - \overline{z} \right)^2}{N-1} \right]^{0.5}$ is the RMS value for all N points together;

$\overline{z}$isthe arithmetical mean value of all N points together;

**) – $f_{cm,cube}$=$f_{ck,cube}$/(1-1,64V_c);

***) – $f_{ck,prism}$=$f_{cm,cube}$(0,77-0,001$f_{cm,cube}$).

For averaging of the obtained data, the statistical processing of the obtained results was carried out, in which the coefficient of variation for samples of cubes amounted to $\nu_c = 10{,}8\%$ and $\nu_c = 13{,}3\%$ for prisms, allowing the value of the concrete class corresponding to the arithmetical mean of experimentally obtained data.According to the table.3.1 DBN V. 2.6-98:2009 [1] we accept the experimentally established class of experimental samples of concrete C30/35.

Processing of results of measurement instruments (indicating gages and electric resistance strain gauges) have allowed to establish experimental dependence σ_c–ε_c in the measured range (Fig.3.3...3.12).

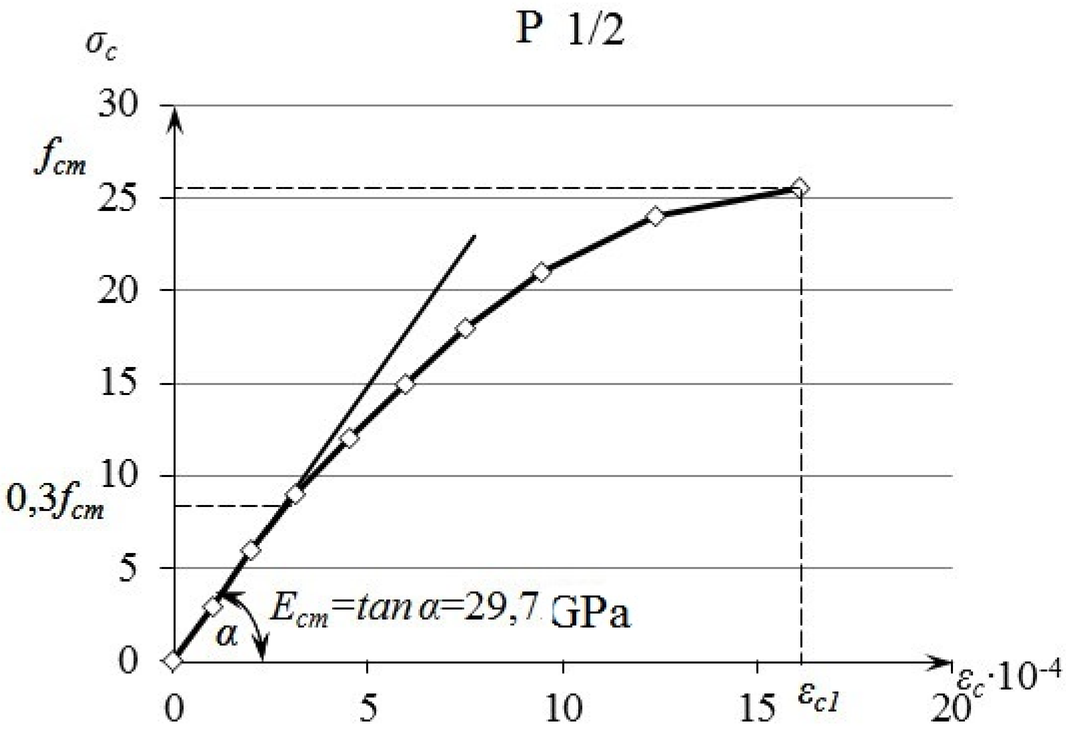


Fig.3.3. Experimental dependence σ_c–$\varepsilon_c$for prism P 1/2.

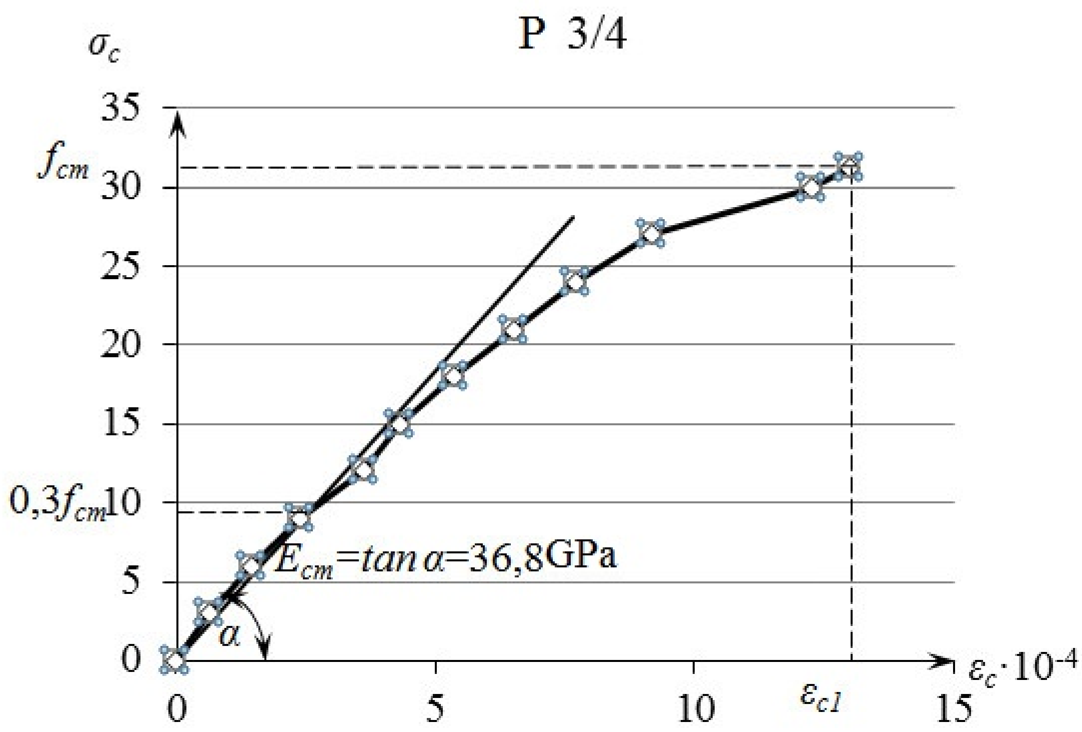


Fig.3.4. Experimental dependence σ_c–$\varepsilon_c$for prism P 3/4.

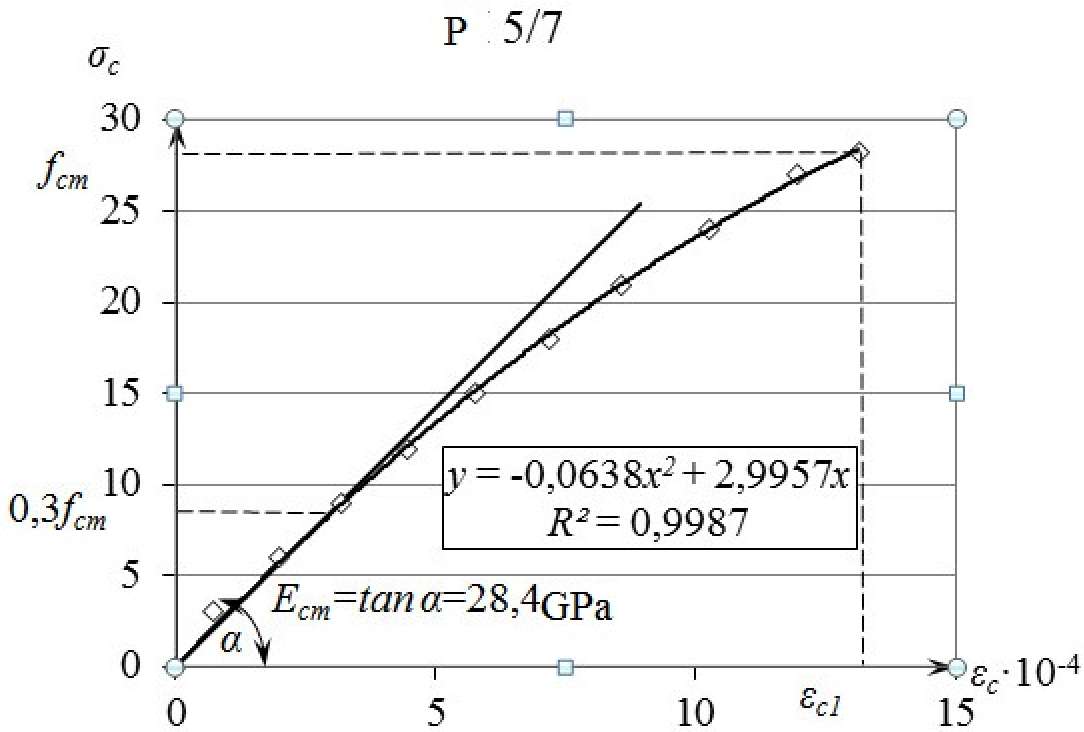


Fig.3.5. Experimental dependence σ_c–$\varepsilon_c$for prism P 5/7 with drawing a trend line.

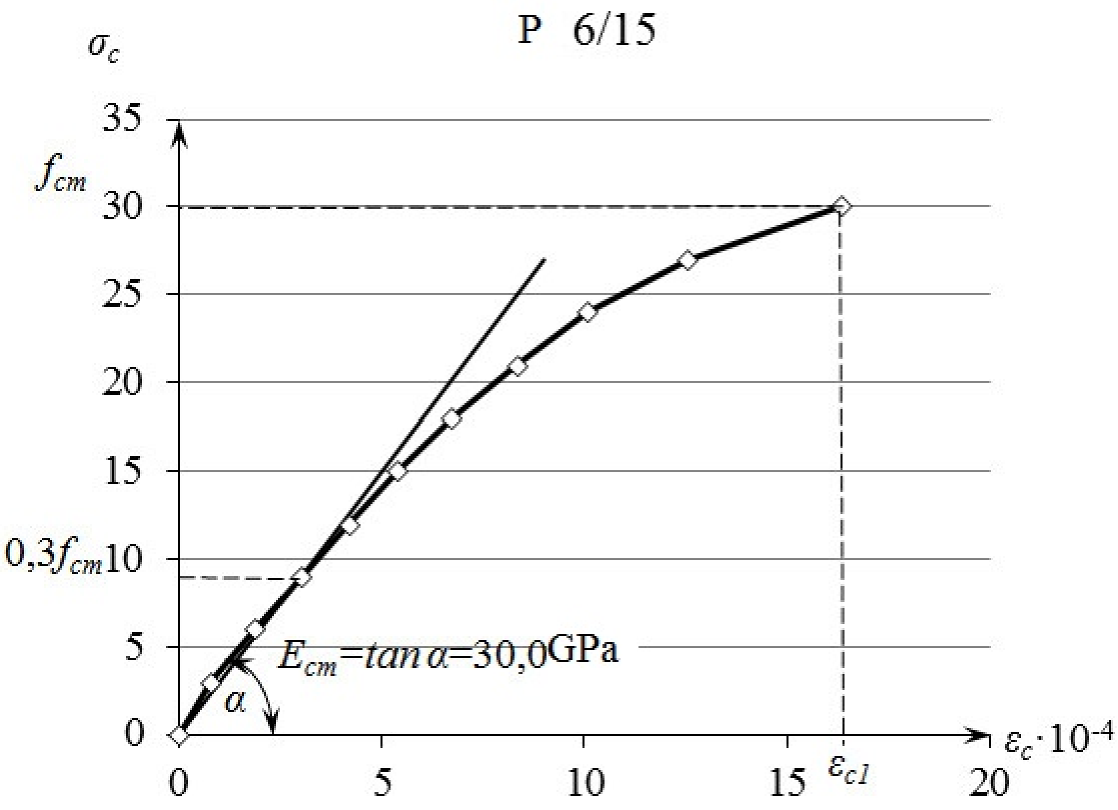


Fig.3.6. Experimental dependence σ_c–$\varepsilon_c$for prism P 6/15.

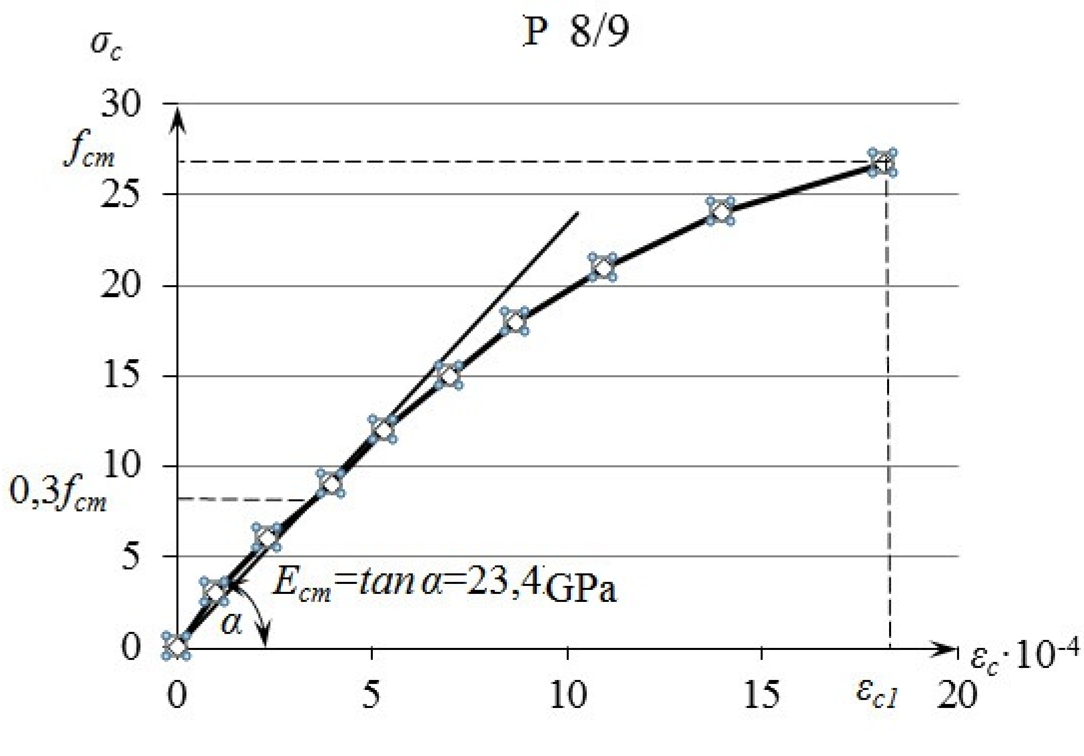


Fig.3.7. Experimental dependence σ_c–ε_c for prism P 8/9.

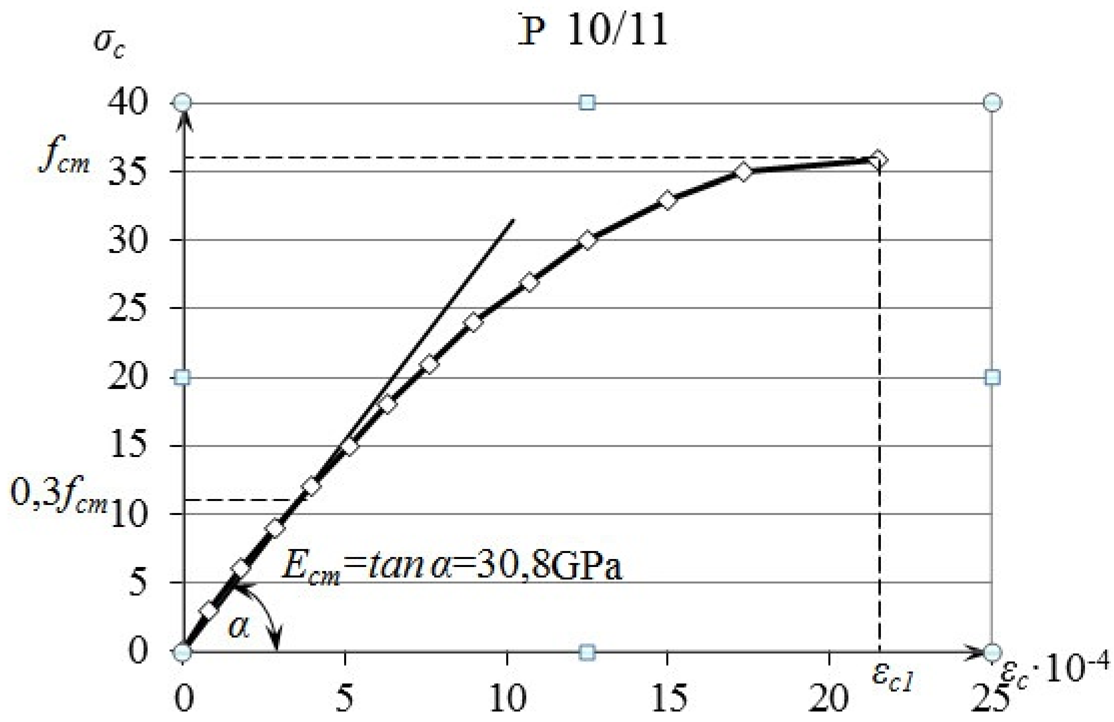


Fig.3.8. Experimental dependence σ_c–$\varepsilon_c$for prism P 10/11.

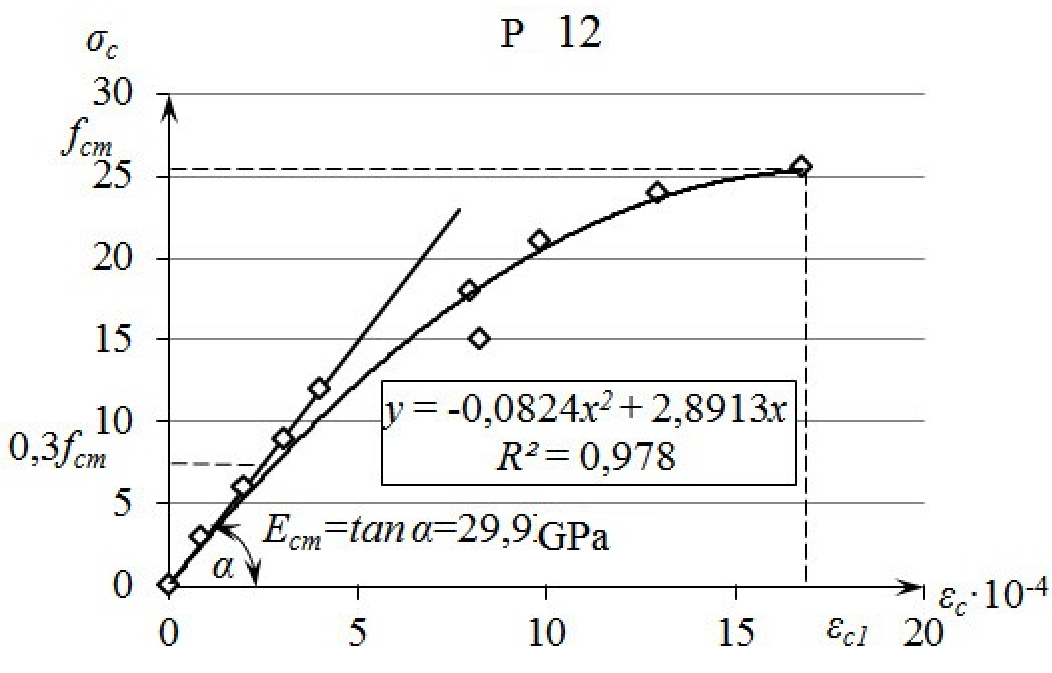


Fig.3.9. Experimental dependence σ_c–ε_c for prism P 12 with drawing a trend line.

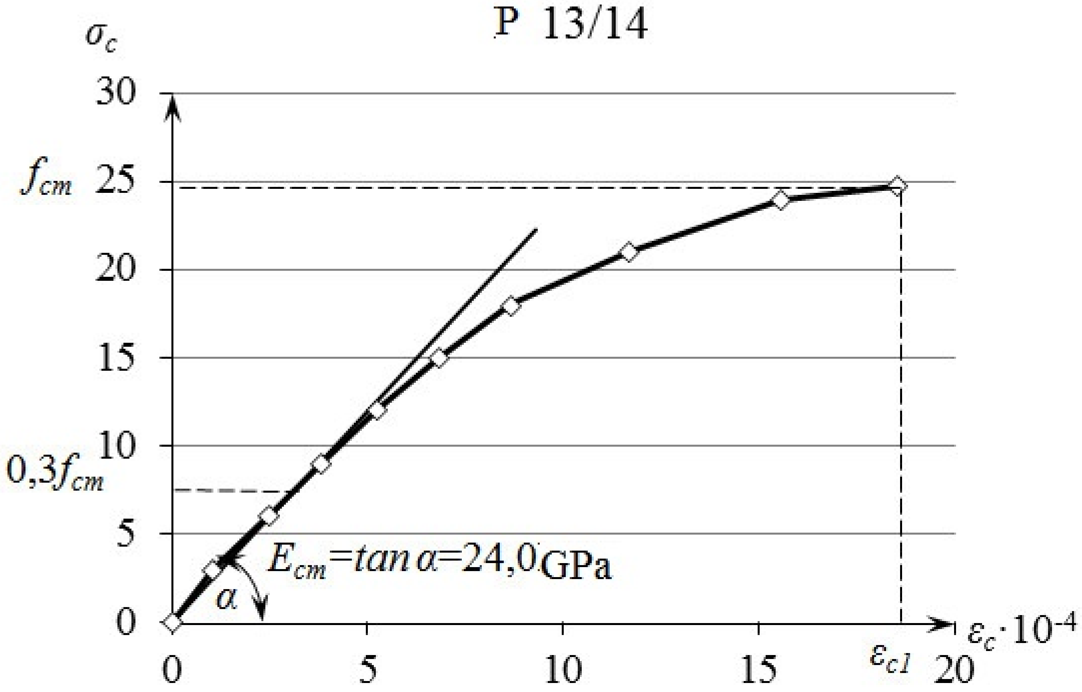


Fig. 3.10. Experimental dependence σ_c–ε_c for prism P 13/14.

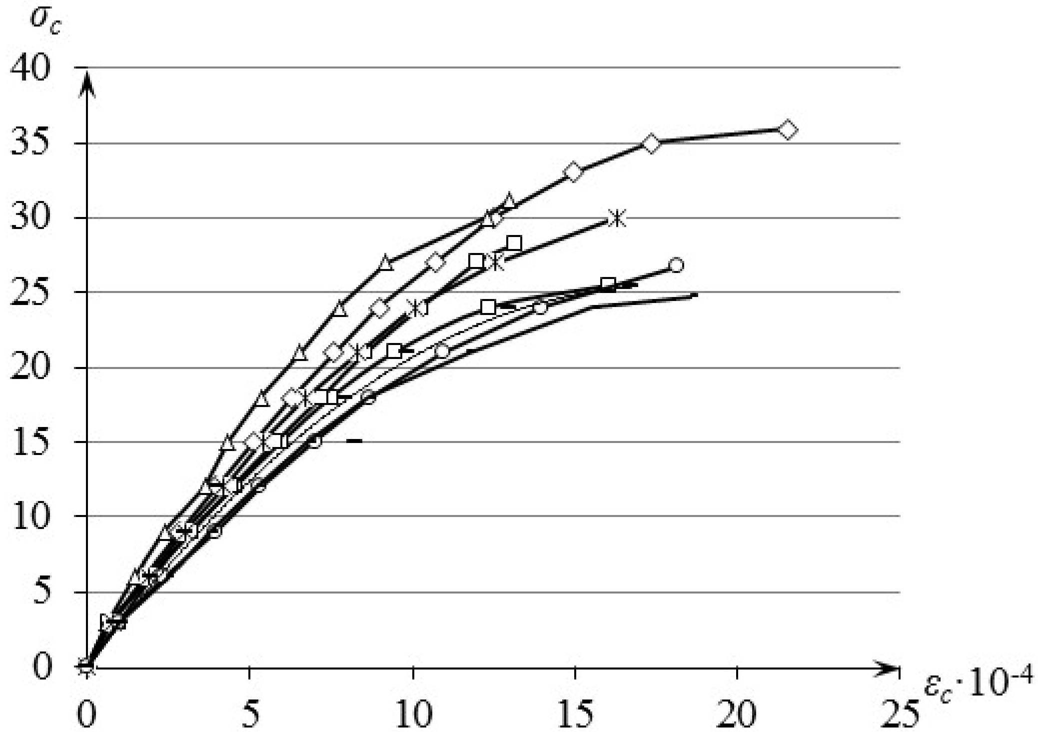


Fig. 3.11.Combined graph of dependence σ_c–ε_c for eight series of prisms.

As the results of testing of reinforcing bars the following characteristics of the working reinforcement were set (table 3.3): yield strength f_{yd} =560 MPa, the tensile strength f_u= 665 MPa; the modulus of elasticity of reinforcement E_s= 2.05×10^5 MPa, relative elongation δ = 22%.

Table 3.3

Mechanical properties of reinforcing steel

Class of the reinforcement	The diameter of the bar, mm	Yield stress f_{yd}, MPa	Ultimate tension f_u, MPa	Elongation tear, δ, %	The modulus of elasticity E_s ×10^5, MPa
1	2	3	4	5	6
A500C	16	594	699	20.0	2.07
		566	679	18.5	2.06
		560	665	22.0	2.05

3.1.2. The results of testing of experimental samples – damaged T-shaped beams. The destruction of experimental samples of 1 series (samples with a width of shelves of 400 mm) B 3, 6, 12, 13 had the same nature (Fig.3.12 … 3.15). The first cracks appeared in pure bending zone at the level of loading (0.25 ... 0.3) F_{ULS}. With increasing load in the span of the cut the inclined cracks began to appear (almost at the same time there were 3 ... 4 inclined cracks when the load value (0.4 ... 0.5) F_{ULS}), which in a further increase in load evolved to the level of the shelf, and often crossed it. The spread of inclined cracks and destruction of concrete in the shelf occurs on a limited surface within the width of the shelf. This surface starts from the edge and opens onto the top face of the shelf, i.e., there is a sort of tripartite forcing the shelf with the edge. In addition, in these samples with wide shelves (400 mm) shortly before the destruction the vertical longitudinal cracks are formed at the junction of the overhangs of the shelf to the edge. Normal cracks as load increases apply to the entire height of the edge and pass on the shelf. The nature of the destruction of this series corresponds to the destruction of the reinforced element when the tension in the tensed reinforcement has not yet reached its yield strength, and the destruction was due to the crushing of concrete of the compressed zone on the predominant action of the bending moment.

In all samples, with the width remaining after injury of the beam by the shelf of 235mm first crack (normal), in contrast to the first series of beams, appeared in the place of application of concentrated forces at the load value (0.18 ... 0.28) F_{ULS}.

When the load increased up to (0.35 ... 0.5) F_{ULS} in the pre-support zones the emergence and further development of the inclined cracks was observed. Beams of this series B 4, 7, 10, 11, 14, 15 were destroyed because of the crushing of concrete of the compressed zone at the area of action of maximum bending moment.

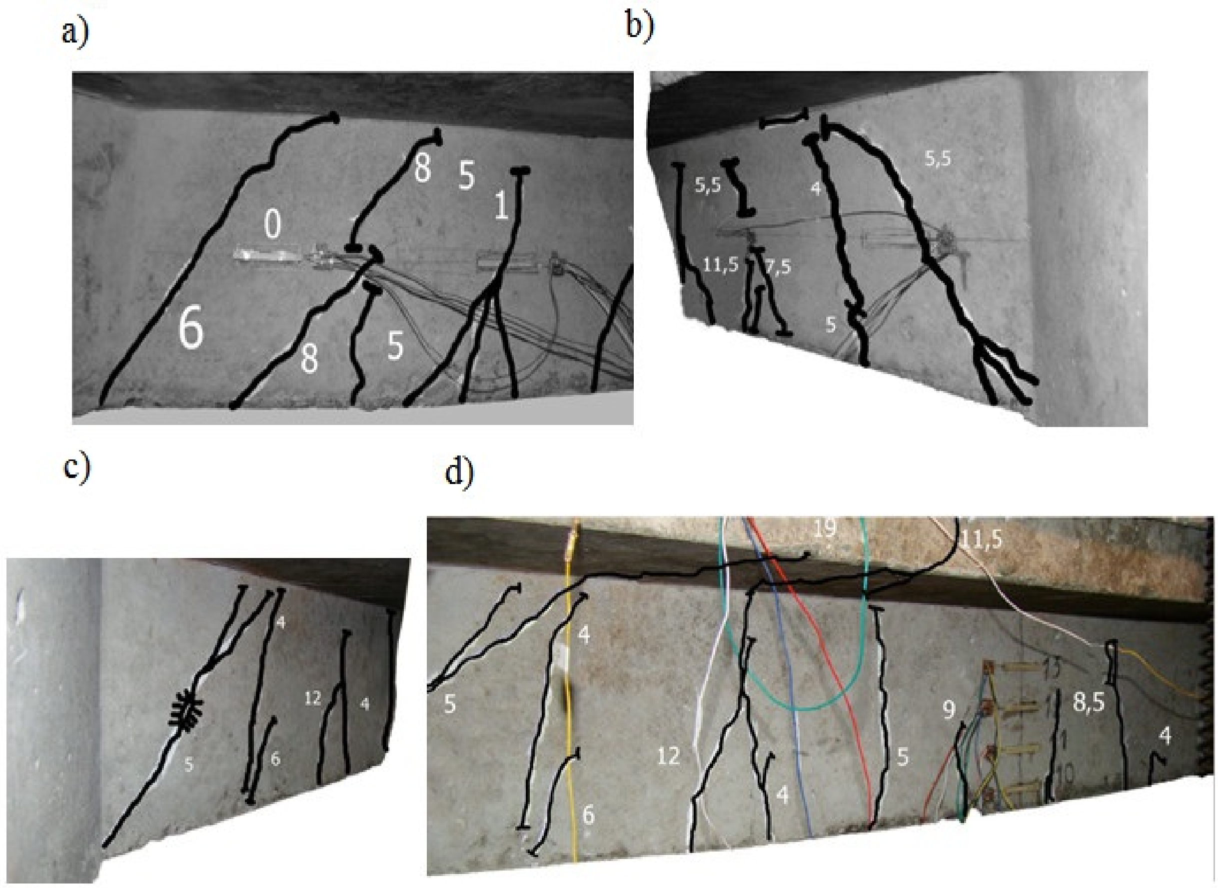


Fig.3.12. Series 1. The destruction of the beam B3: a - A view of the left support; b - A view of the right support; c - B view of the left support; d - B view of the central section.

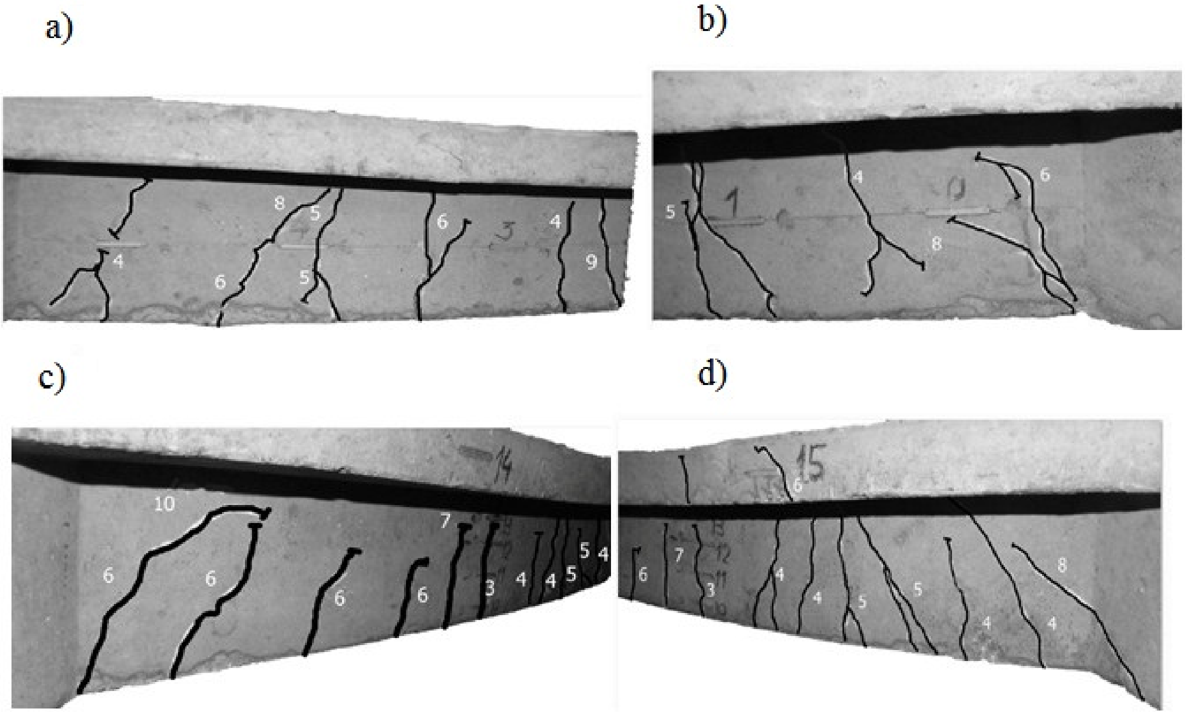


Fig.3.13. Series 1. The destruction of the beam B6: a - A view of the left support; b - A view of the right support; c - B view of the left support; d - B view of the right support.

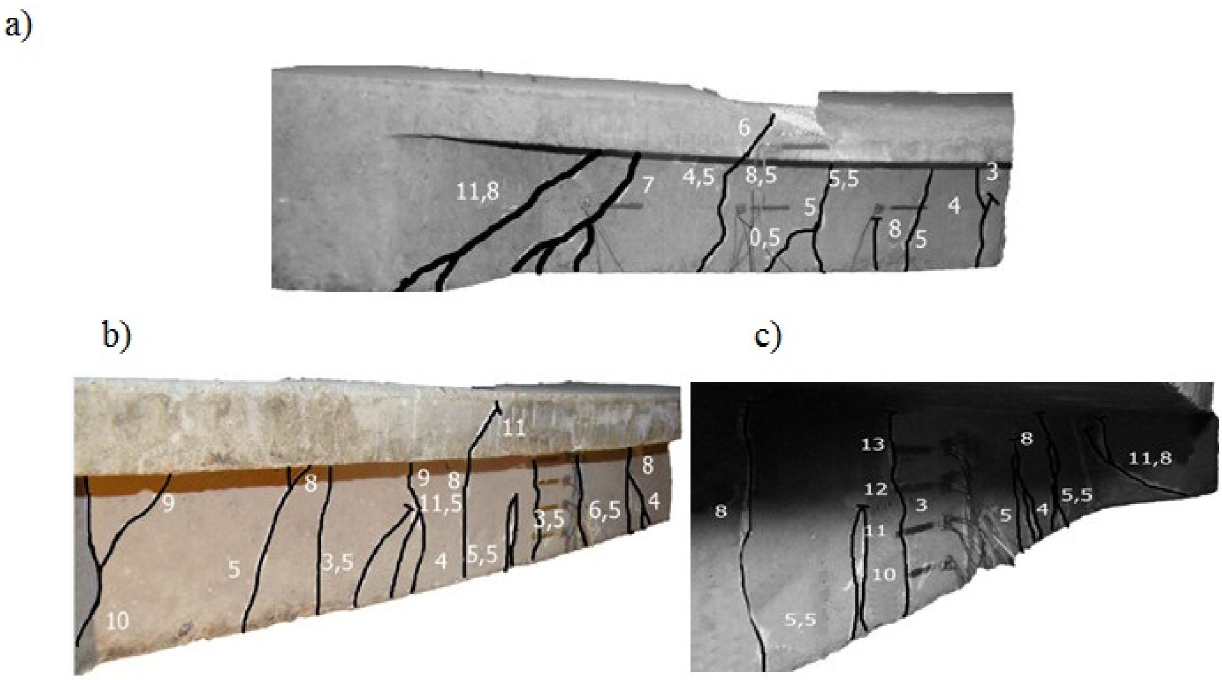


Fig.3.14. Series 1. The destruction of the beam B12: a - A view of the left support; b - B view of the left support; c - B view of the right support.

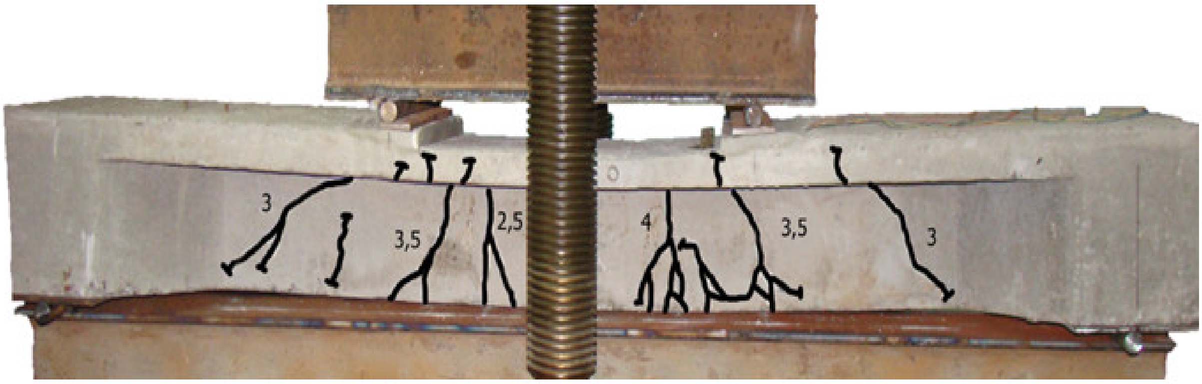


Fig.3.15. Series 1.The destruction of the beam B13 – B view.

In the majority of the samples in this series there were significant dismantling of the protective layer of concrete of the stretched zone of the beam and cracking of the concrete of compression zone within the area of the damage of the beam. The destruction of the sample corresponds to the destruction of the armored reinforced concrete bent elements and is characterized by underutilization of the mechanical properties of tensile reinforcement.

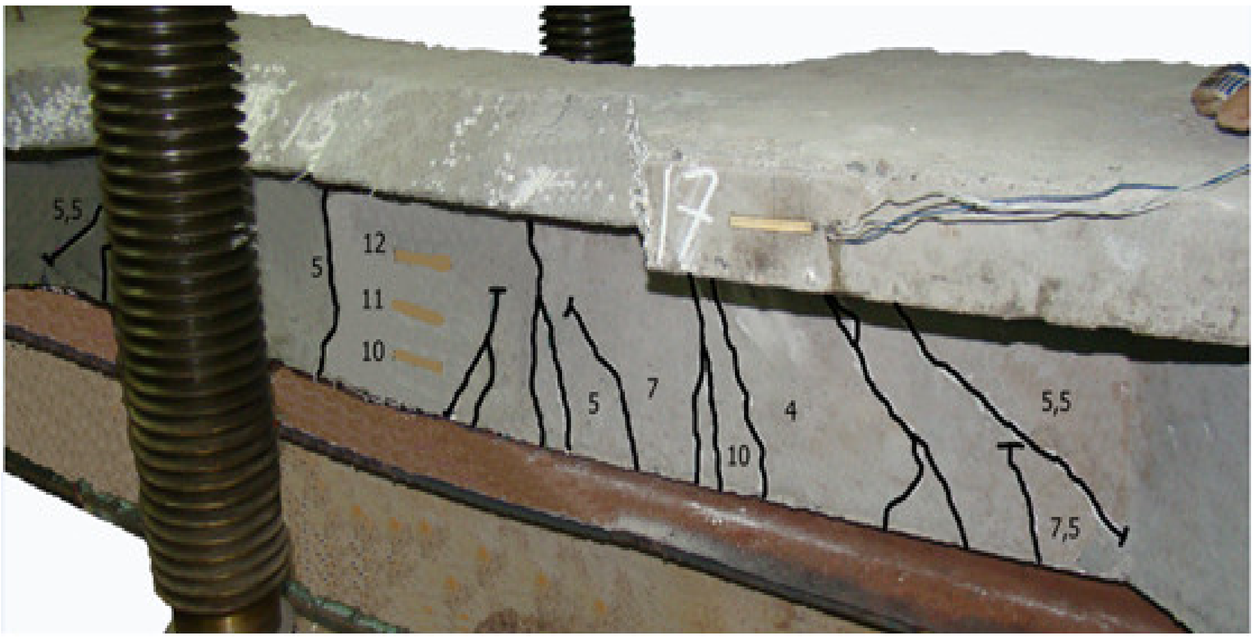


Fig.3.16. Series 2.The destruction of the beam B4 – B view.

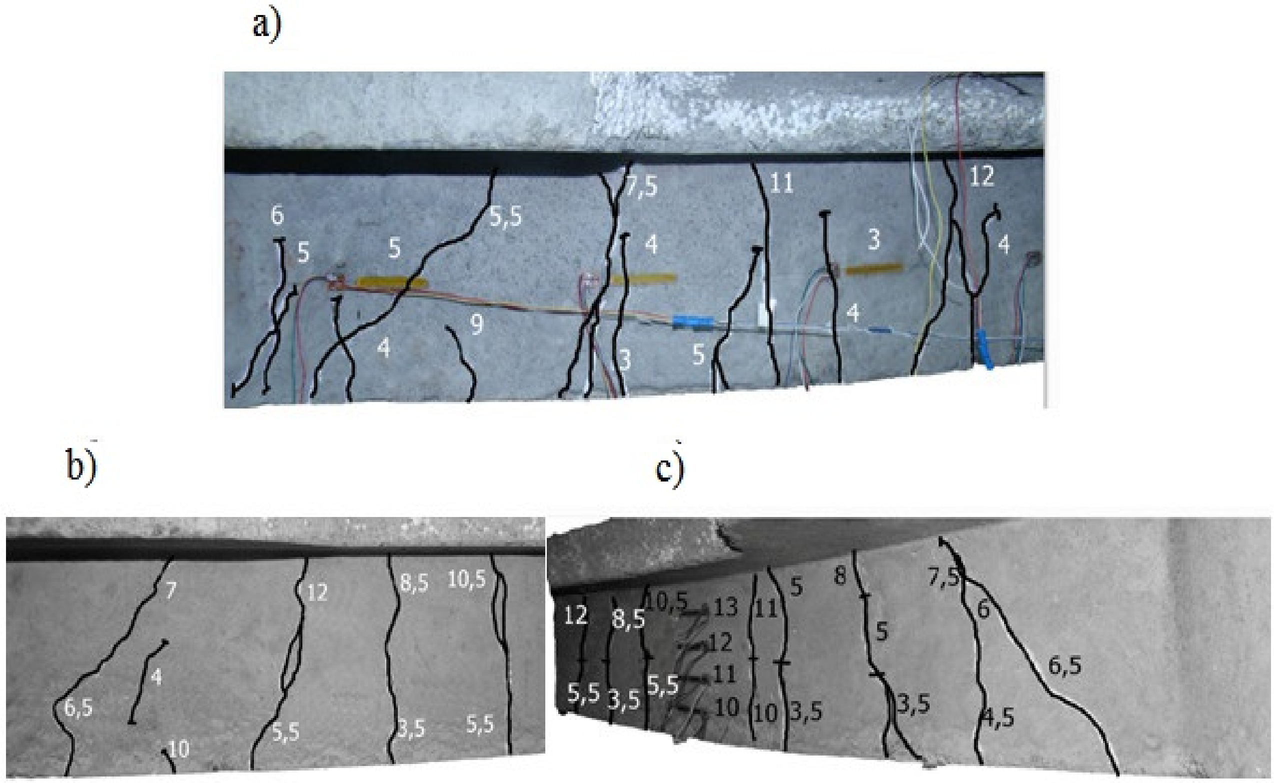
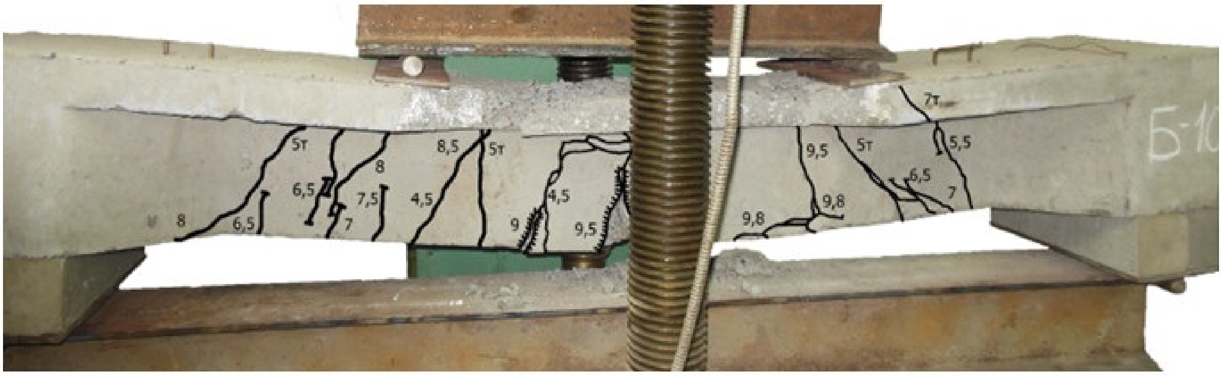


Fig.3.17. Series 2. The destruction of the beam B7:
a - A view of the left support and the central section; b - B view of the left support;
c - B view of the right support.

In the beams of the third series B 1, 2, 5, 8, 9, which in the zone of pure bending had almost no shelves, at the loading level of (0,2...0,3) F_{ULS}S first appeared inclined cracks, and then normal ones. The destruction of beams of this series was mostly fragile.

Normal cracks that appeared in the zone of pure bending of beams, with a slight increase in the load were rapidly distributed throughout the height of the beam section.

a)



b)

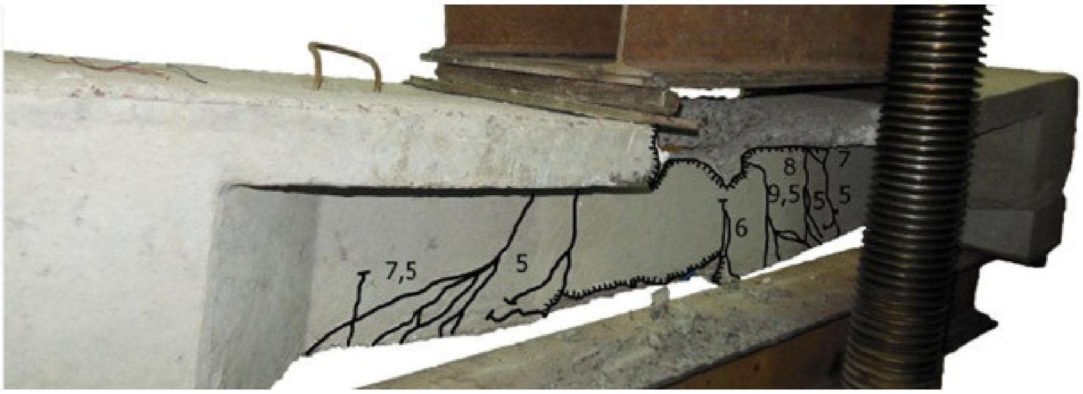
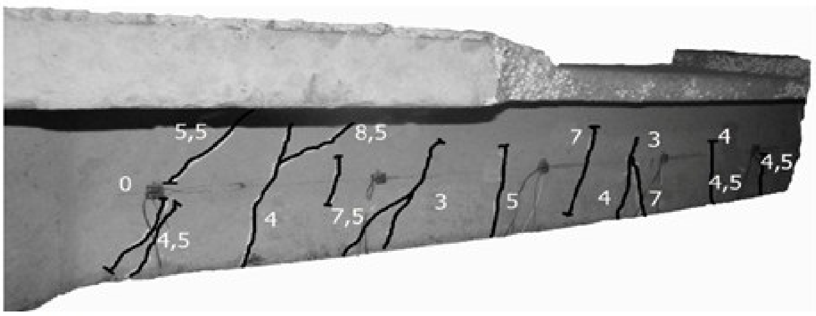


Fig.3.18. Series 2. The destruction of the beam B10: a - A view; b –B view.

a)



b)

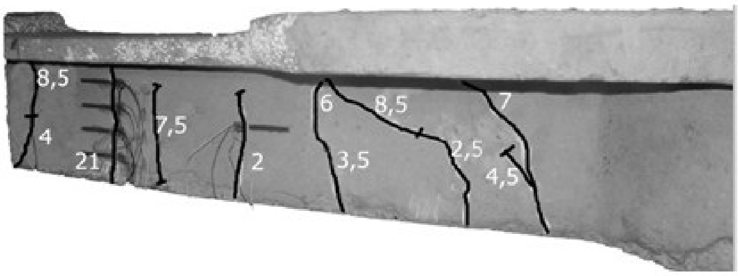


Fig.3.19. Series 2. The destruction of the beam B11: a - A view of the left support and the central section; b - B view of the right support and the central section.

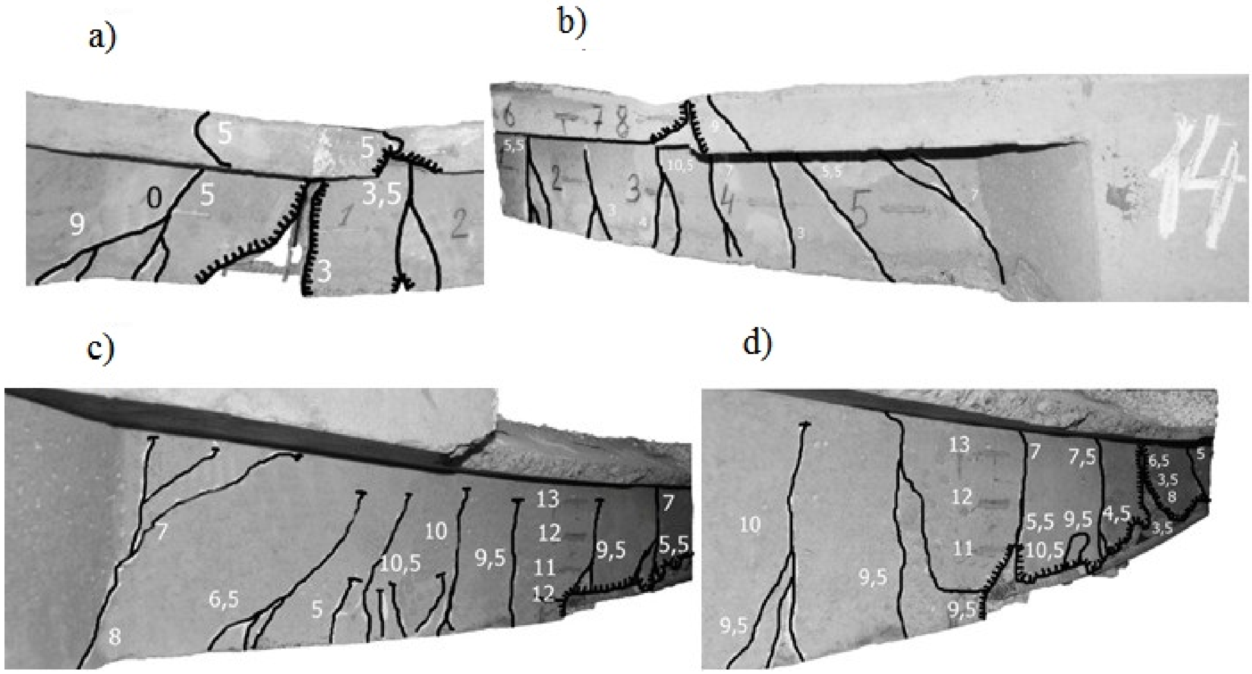


Fig.3.20. Series 2. The destruction of the beam B14: a - A view of the left support; b - A view of the right support; c – B view of the left support; d - B view of the central section and fragment of the right support.

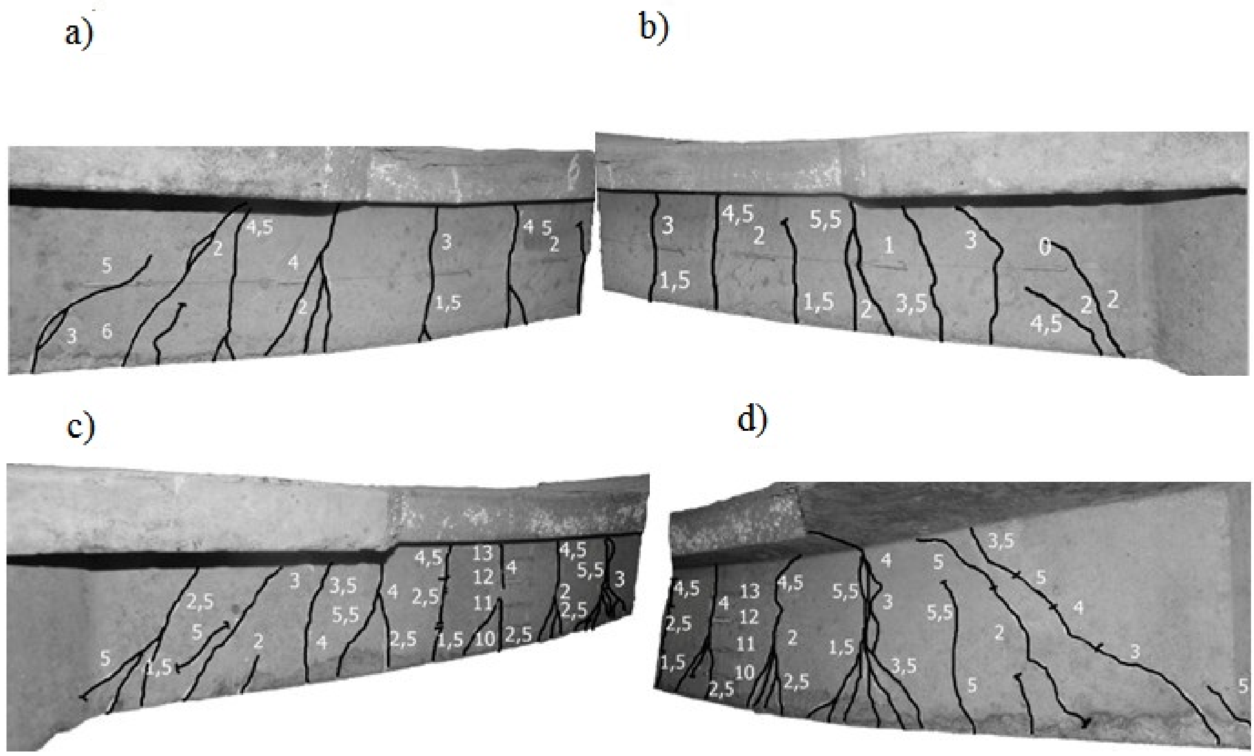


Fig.3.21. Series 2. The destruction of the beam B15: a - A view of the left support; b - A view of the right support; c – B view of the left support; d - B view of the central section and fragment of the right support.

Inclined cracks that occurred in the span of the cut of the beams, hardly reached the middle height of the beam. There was also observed the formation of longitudinal cracks in the tensile zone of the concrete due to a significant increase in deflection of beams and, as a consequence, cracking of the protective layer of concrete of the stretched zone. Along with this, the crushing of the concrete of the compressed zone within the area of damage of the beam occurred.

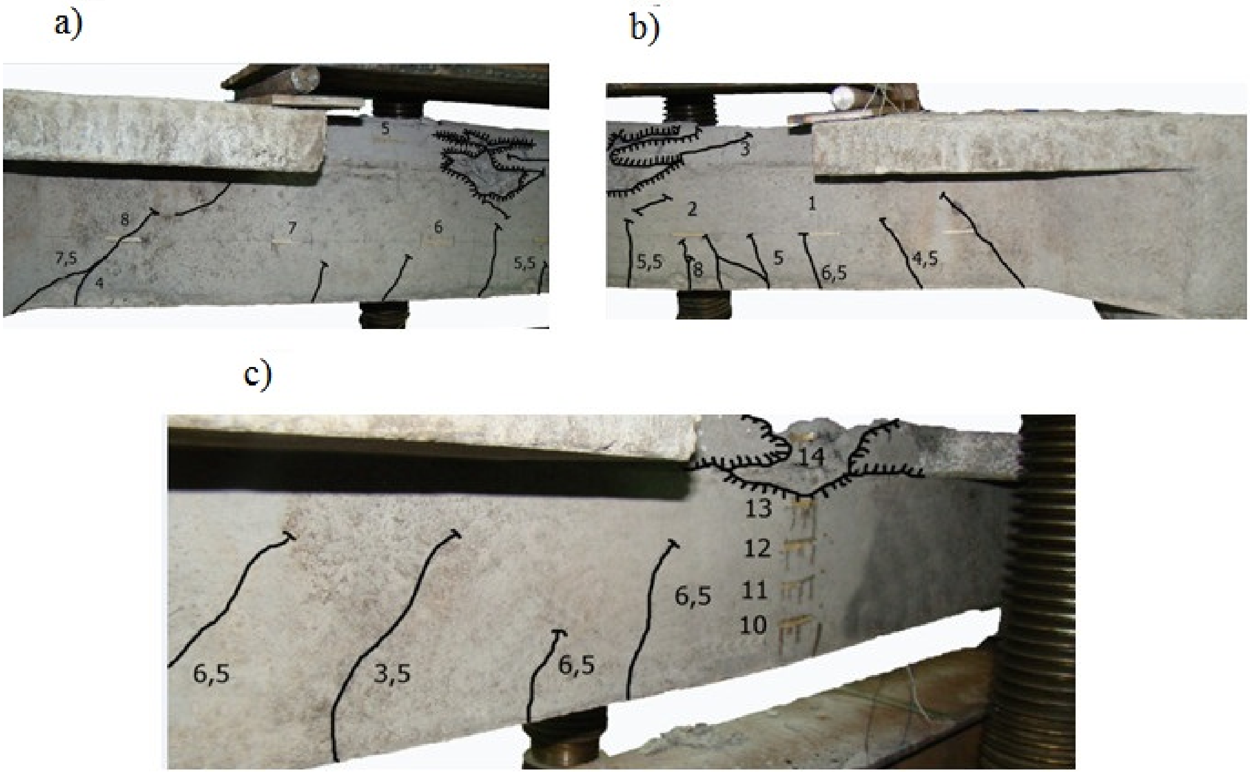


Fig.3.22. Series 3. The destruction of the beam B1: a - A view of the left support; b - A view of the right support; c – B view of the left support and the central section.

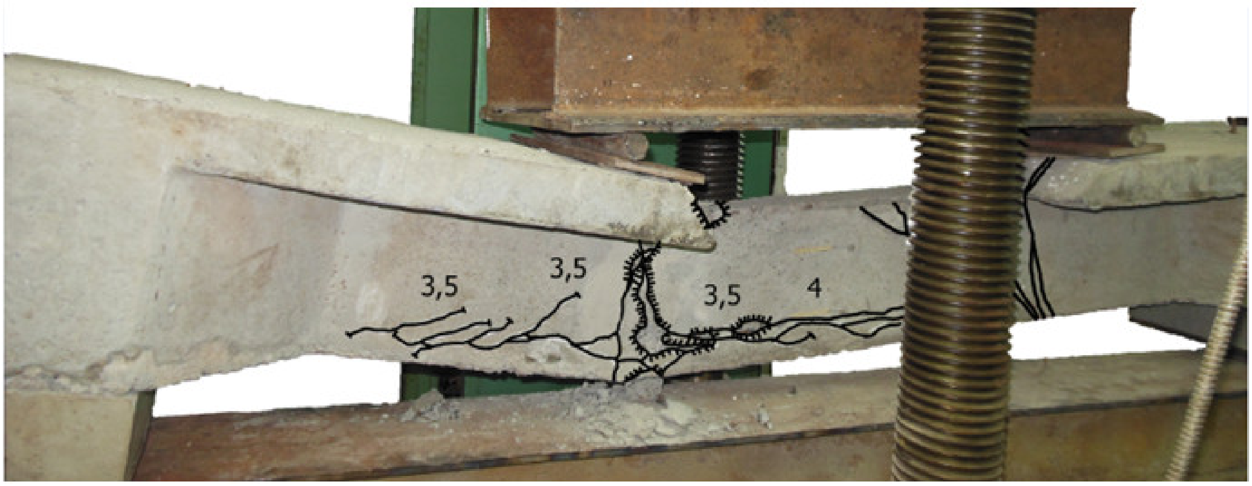


Fig.3.23. Series 3.The destruction of the beam B2 - A view.

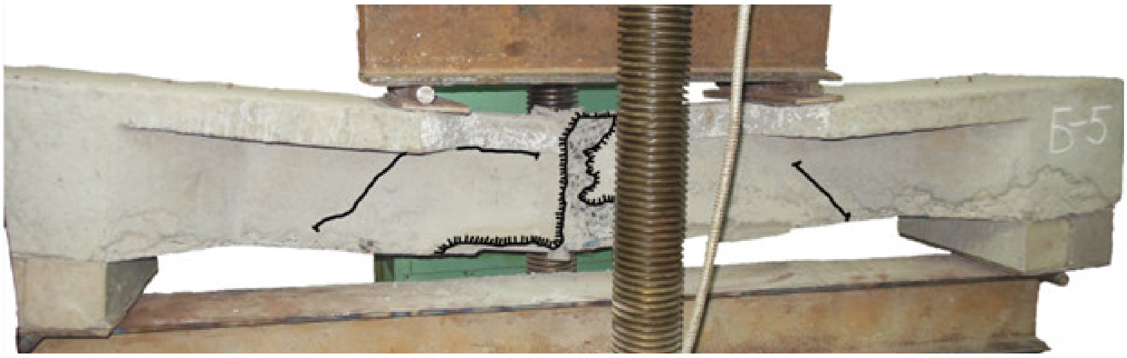


Fig.3.24. Series 3.The destruction of the beam B5 - A view.

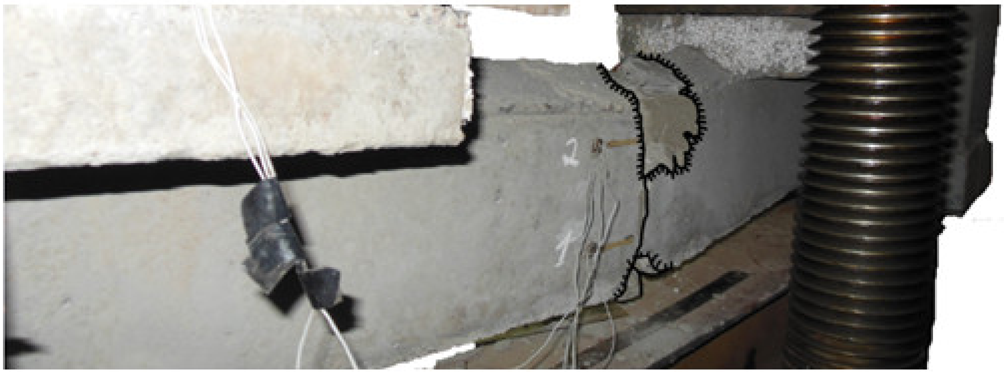
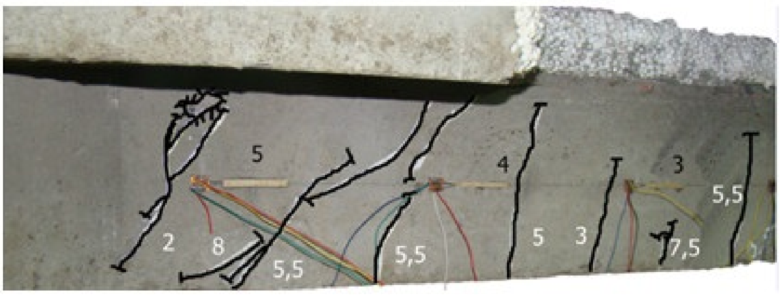
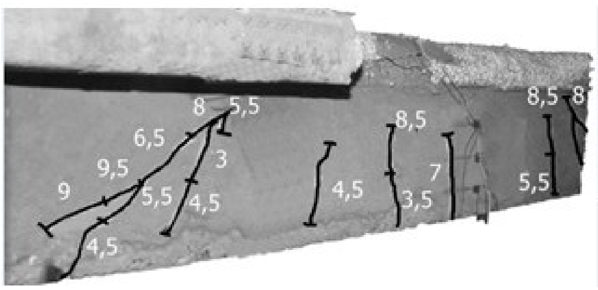


Fig.3.25. Series 3.The destruction of the beam B8 - B view.

a)



b)



c)

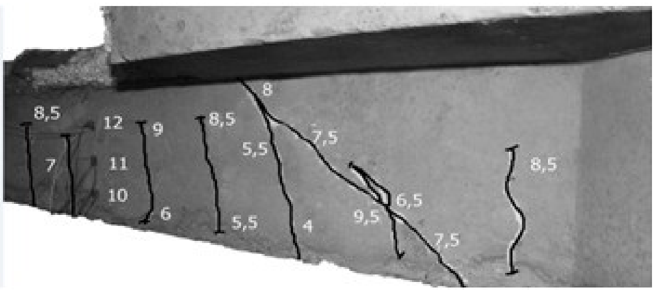


Fig.3.26. Series 3. The destruction of the beam B9: a - A view of the left support; b - B view of the left support; c – B view of the right support.

When conducting the experimental studies the values of external load were fixed, that were corresponding to the appearance of the first normal cracks $F_{w,ult\perp}$ in the zone of pure bending of the samples, and also inclined cracks $F_{w,ult/}$ in the spans of the experimental beams and destruction of beams F_{ULS} (Table 3.2) [129].

Table 3.2

The values of the external loads corresponding to the appearance of the first normal, inclined cracks and destruction of the beams

Label of the beam	$F_{w,ult\perp}$, kN	$F_{w,ult/}$, kN	F_{ULS}, kN	M, kNm
B 1	55	35	80	20
B 2	40	35	40	10
B 3	40	50	130	32,5
B 4	20	40	105	26.25
B 5	30	25	95	23.75
B 6	30	50	130	32.5
B 7	30	35	110	27.5
B 8	30	25	75	18.75
B 9	30	20	95	23.75
B 10	25	50	98	24.5
B 11	20	30	90	22.5
B 12	30	50	118	29.5
B 13	25	30	90	22.5
B 14	30	50	105	26.25
B 15	20	40	110	27.5

By analyzing the data obtained experimentally it is seen that the greatest load was withstood by the samples that had the least damage and the least was withstood by samples with the width of the damaged shelf b_{eff} = 70 cm. A distinctive feature of the deformation of the beams was different nature of development of cracks up to the operational level of loading.

In the process of processing of the experimental data by the method of [119], with the removal of insignificant coefficients of regression equations and conversion of the remaining coefficients, an adequate mathematical model (3.1) was obtained, which has sufficient information usefulness, and by which it is possible to assess the influence of the studied factors on the output parameters of beams, the geometric interpretation of which is presented in Fig. 3.27...3.28.

By the obtained experimental data for an external load a nonlinear experimental-statistical model (ES-model, the formula 3.1) was constructed, that is adequate to the experiment with an error $s_e\{lnF_{ULS}\}$=0,058 with 9 statistically significant coefficients.

$$
\begin{aligned}
ln\{F_{ULS}\} = {} & 4,73 + 0,10x_1 - 0,21x_1^2 + 0,20x_1x_2 + 0,07x_1x_3 \\
& - 0,13x_2 \pm 0\,x_2^2 - 0,11x_2x_3 \\
& - 0,15x_3 - 0,06x_3^2
\end{aligned}
\tag{3.1}
$$

The data from the ES-model (3.1) can be analyzed by one-factor dependencies shown in Fig. 3.27. The impact of three factors on the analyzed criterion of the quality is built so that they pass through the extreme points of the *min* and *max*.

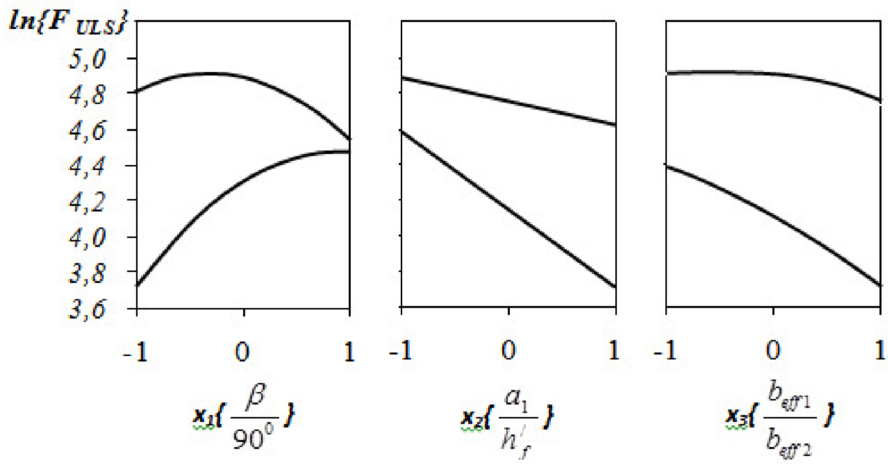


Fig. 3.27.One-factor dependencies of the influence of varied factors on the measure of the strength of beams.

Generalizing indicators of the one-factor diagrams are: maximum $F_{ULS.max}$ = 131.50 kN is reached at the point with coordinates x_1= – 0.32, x_2= – 1, X_3= – 0.54 and respectively minimum $F_{ULS..min}$= 40.89 kN – x1= – 1, x_2=X_3= + 1; absolute increment $\Delta\{F_{ULS}\}$ = 90.61 kN and relative increment $\delta\{ F_{ULS} \}$ = 3.2.

The result of mathematical processing of the values of the external load of reinforced concrete beams, by definition of the joint effect of varied factors is presented in Fig. 3.28.

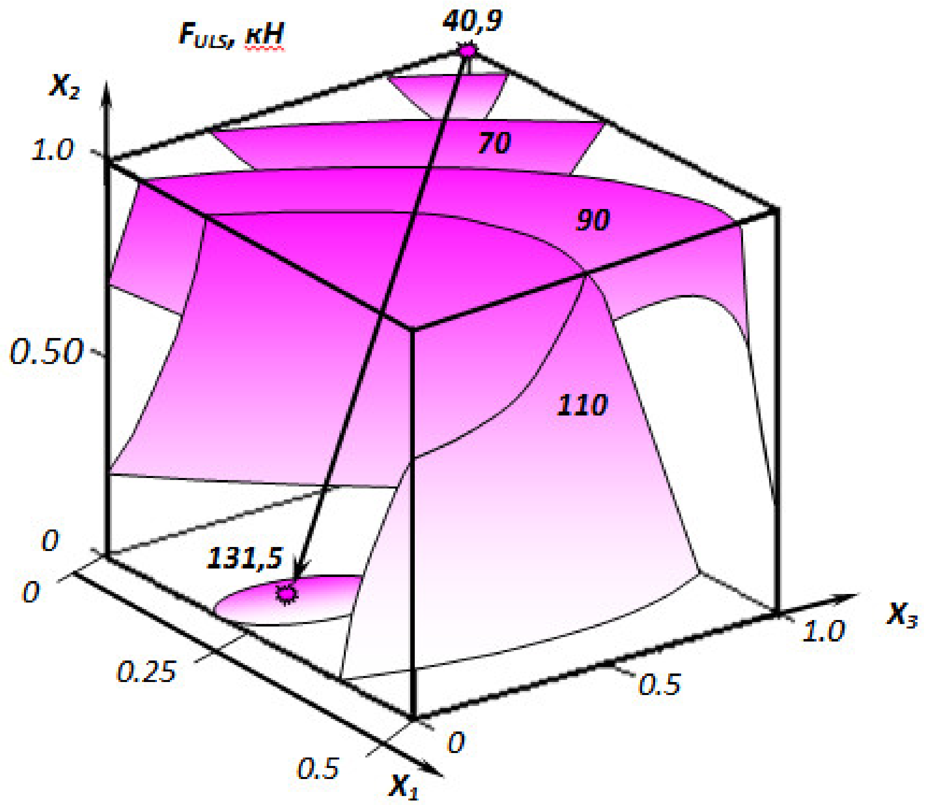


Fig.3.28. Graph of the joint effect of varied factors on breaking load of beams.

The values of the factors of variation in figure 3.28. stated in absolute values.

From the analysis of the presented diagrams it follows that with increase both of the angle of the damage $\boldsymbol{x_1}\{\frac{\beta}{90^0}\}$to 0.5; and the value of damaged part of the shelf $\boldsymbol{x_3}\{\frac{b_{eff1}}{b_{eff2}}\}$ to 1 samples-beams are characterized by the least action of external load on them and accordingly reinforced concrete beam can withstand a maximum external load of 131.5 kN at an angle of damage not more than $15.3^0/90^0$ (X_1=0.17) and the value of damaged part of the shelf no more than X_3=0.23 (X_3=37.95 mm) in the absence of the depth of damage. From the above it can be concluded that the factor having the greatest impact on the bearing capacity of the beams, is the depth of the damage, which is also confirmed by the figure 3.27, where the relation between the change in the depth of damage and breaking load is linear [64].

The deflection of the experimental samples was measured in the zone of pure bending symmetrically on both sides of the beam with the help of indicating gages at each stage of loading. The deflection of beams before the formation of the first cracks was not observed. The emergence of the first normal, and then inclined cracks was accompanied by an abrupt increase of deflection, the relationship between deflection and load is proportional. After the formation of these cracks increase in deflection was not proportional to the increase of external load. By the exhaustion of the bearing capacity of the beam deflection was increased even with a small increase in external loading (Fig. 3.29 … 3.31).

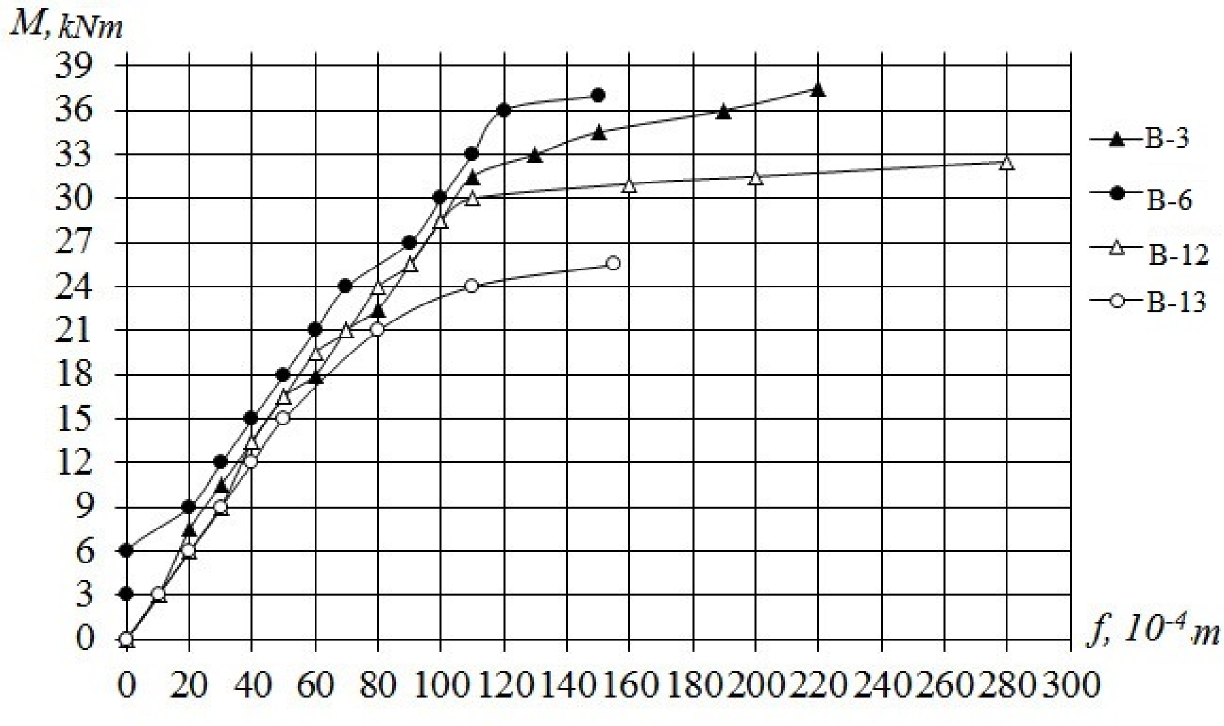


Fig.3.29. The change in the deflection of beams of 1 series with increasing bending moment.

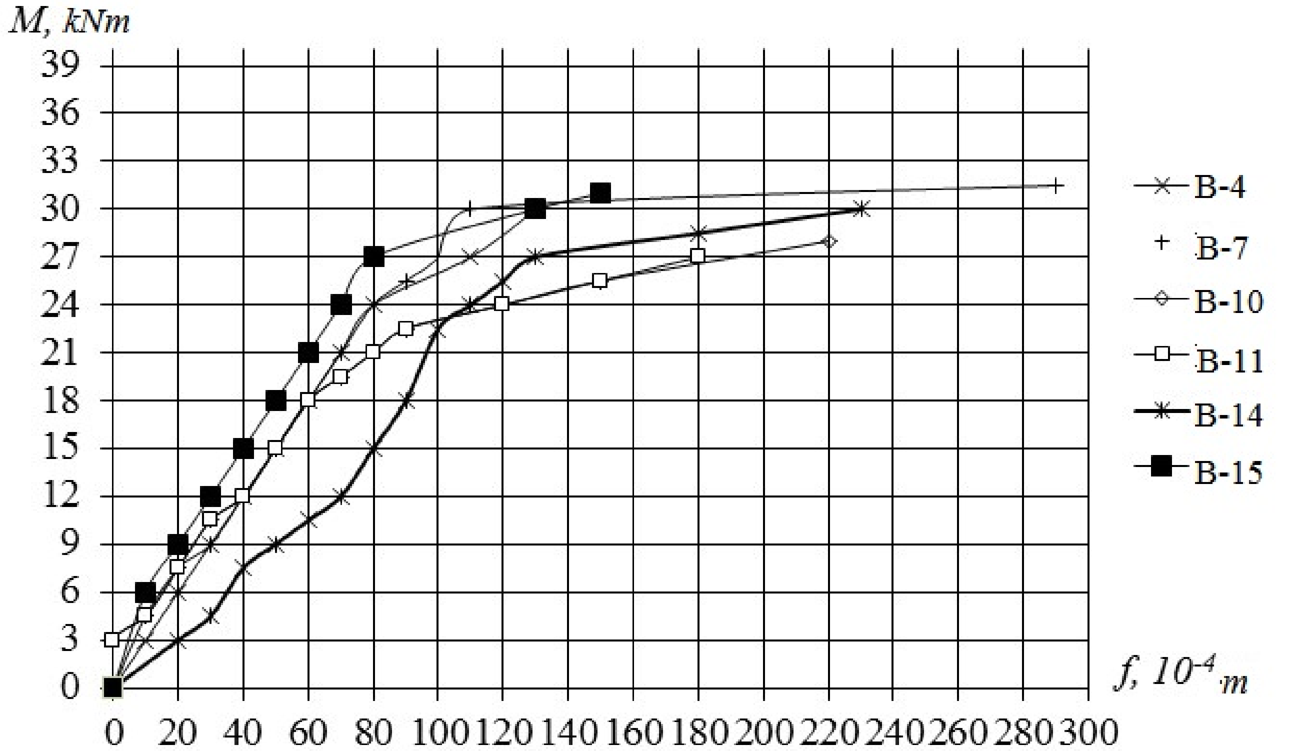


Fig.3.30. The change in the deflection of beams of 2 series with increasing bending moment.

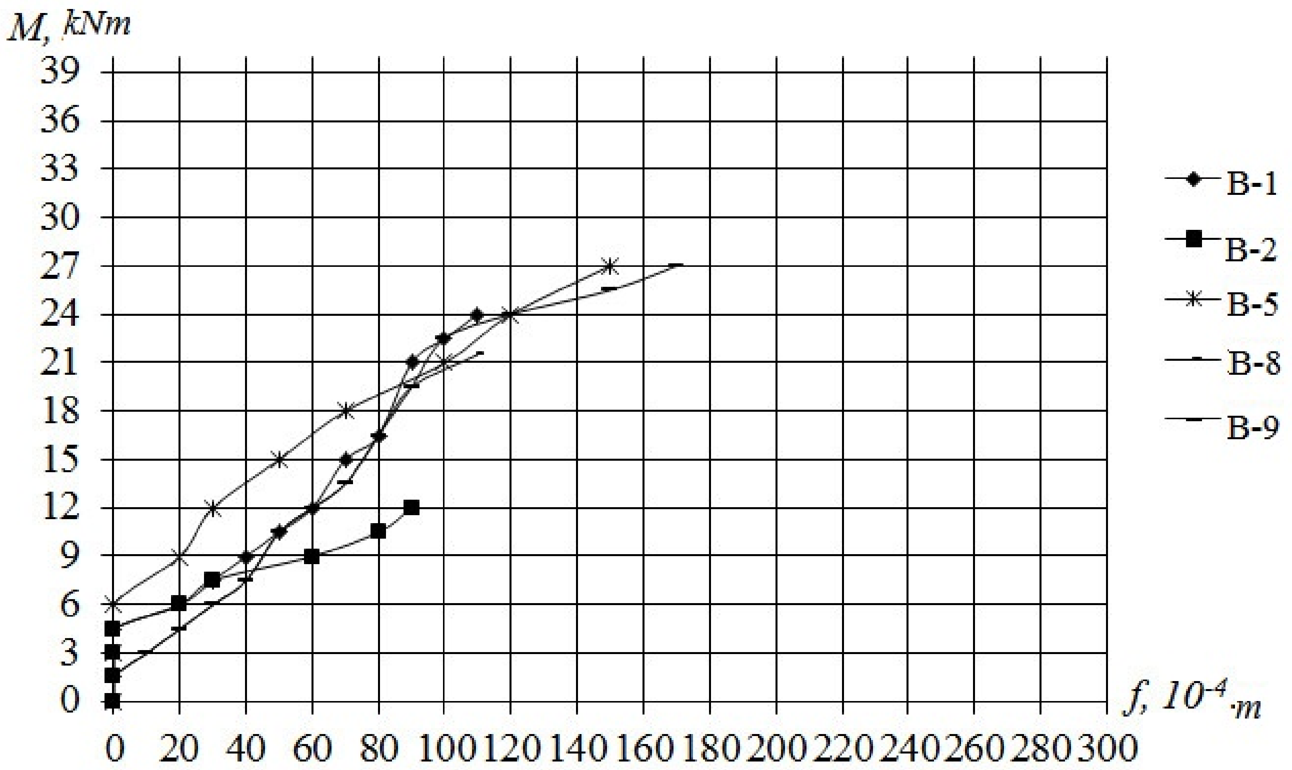


Fig.3.31. The change in the deflection of beams of 3 series with increasing bending moment.

Table 3.3 shows the average values of deflections of the experimental beams in the places of their measurements on "operational" (0.67 F_{ULS}) level of loading and before the destruction(0.95 F_{ULS}).

Table 3.3

The average values of deflections of beams on "operational" (0.67 F_{ULS}) level of loading and before the destruction(0.95 F_{ULS})

Label of the beam	f at 0.67 F_{ULS}, cm	f at 0.95 F_{ULS}, cm	Natural values of the factors		
			$\beta/90^0$, (β,^{0})	$a_1/h_f^/$, (a_1, mm)	b_{eff1}/b_{eff2}, (b_{eff}, mm)
B 1	0.8	1.1	0.5 (45^0)	1 (60)	1 (70)
B 2	0.6	0.9	0 (0^0)	1 (60)	1 (70)
B 3	0.9	2.2	0.5 (45^0)	1 (60)	0 (400)
B 4	0.7	1.3	0.5 (45^0)	1 (60)	0.5 (235)
B 5	0.8	1.5	0 (0^0)	0 (0)	1 (70)
B 6	0.8	1.5	0 (0^0)	0 (0)	0 (400)
B 7	0.8	2.9	0.25 (22.5^0)	0.5 (30)	0.5 (235)
B 8	0.8	1.1	0.25 (22.5^0)	1 (60)	1 (70)
B 9	0.9	1.7	0.5 (45^0)	0.5 (30)	1 (70)
B 10	0.7	2.2	0.5 (45^0)	0.5 (30)	0.5 (235)
B 11	0.6	1.5	0 (0^0)	0.5 (30)	0.5 (235)
B 12	0.8	2.8	0.25 (22.5^0)	0.5 (30)	0 (400)
B 13	0.7	1.55	0 (0^0)	0.5 (30)	0 (400)
B 14	0.9	2.3	0.25 (22.5^0)	1 (60)	0.5 (235)
B 15	0.65	1.5	0 (0^0)	0 (0)	0.5 (235)

By analyzing the obtained values of deflections it was found that the largest deflections were of the beams B 7, B 12, B 14, the distinctive feature of which is the angle of damage $\beta=22.5^0$. The lowest recorded deflections were of the beams of series 3 - B 1, B 2, B 8, with the width of the damaged shelf b_{eff}= 700 mm.

3.2. The analysis of stress-strained state of experimental samples - damaged T-shaped beams

3.2.1. The analysis of the development of deformations in concrete. The adopted methodology of the research allowed to record the deformation of concrete and reinforcement (with the help of electric resistance strain gauges) during the tests, and to use this data to get graphs of the distribution of deformations depending on loading level.

During short-term loading of the beams the following distribution of strains in concrete and reinforcement was recorded depending on the load level (Fig.3.32).

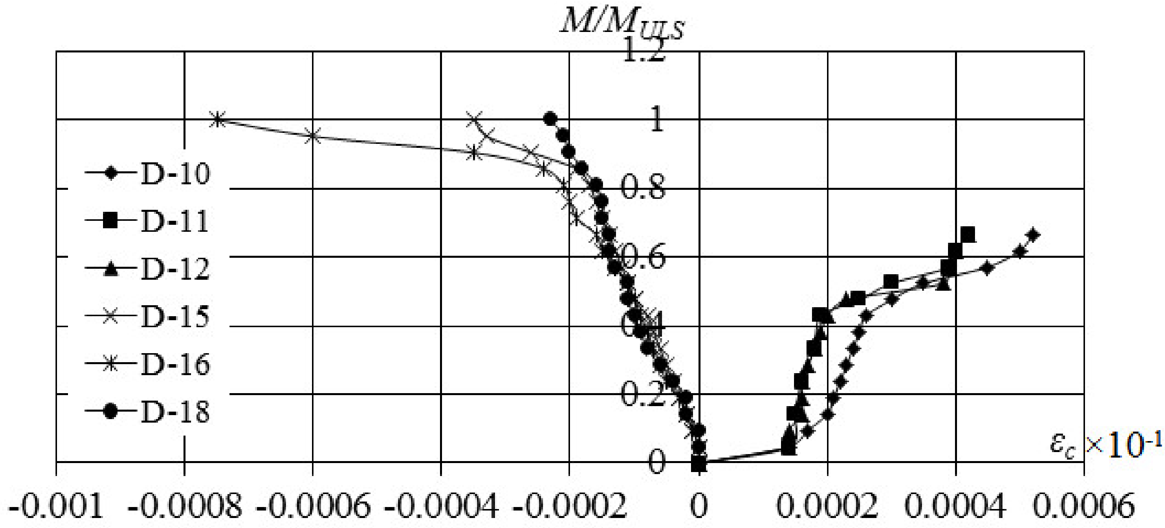


Fig. 3.32.The distribution of longitudinal strains in a beam 4 B in the height of the sample (gauges 10,11,12 – in edge; 15,16,18 – in shelf).

On the example of the beam B 4 it is seen that at the load level of (0.1...0.5) M_u in the edge of the beam the tensile strain development is observed (gauges 10, 11, 12), dependence "loading – deformation" is linear; with further increase of the load corresponding to the crack formation in the zone of location of the gauges, the deformation in the edge begin to grow rapidly, the linear relationship is violated due to plastic deformations of the concrete and further development of cracks and formation of new ones. The shelf of the beam is in a compressed state, as evidenced by the strain gauges 15, 16, 18, a sharp increase of deformation is observed in the shelf before the destruction of the beam at the level of loading (0.8...1) M_u, that is, crushing of the compressed zone of concrete occurs.

A similar pattern of deformation of the beams along the height was observed in all samples. The position of the height of compressed zone of concrete can be monitored by considering the development of deformations in the samples, for example, in the sample B 13.

All the sensors presented on Fig. 3.33 are in the edge. The most deformed are the lower layers of concrete (gauges 6, 5, 4, 3, 2), and up the edge the deformations of the extension decrease (gauge 1) as well as the development of compressive deformations (gauge 0), i.e. the boundary of the height zone of the concrete is level with the top of the edge between gauges 0 and 1.

For a more complete analysis of the deformed state the gauges were located in two ways:

– on the height of the beam (from bottom of concrete to top in the zone of maximum bending moment);

– on the length of the beam (from one support to another on the level of the center of the edge).

Considering the development of deformations along the length of the beams, a number of features were identified.

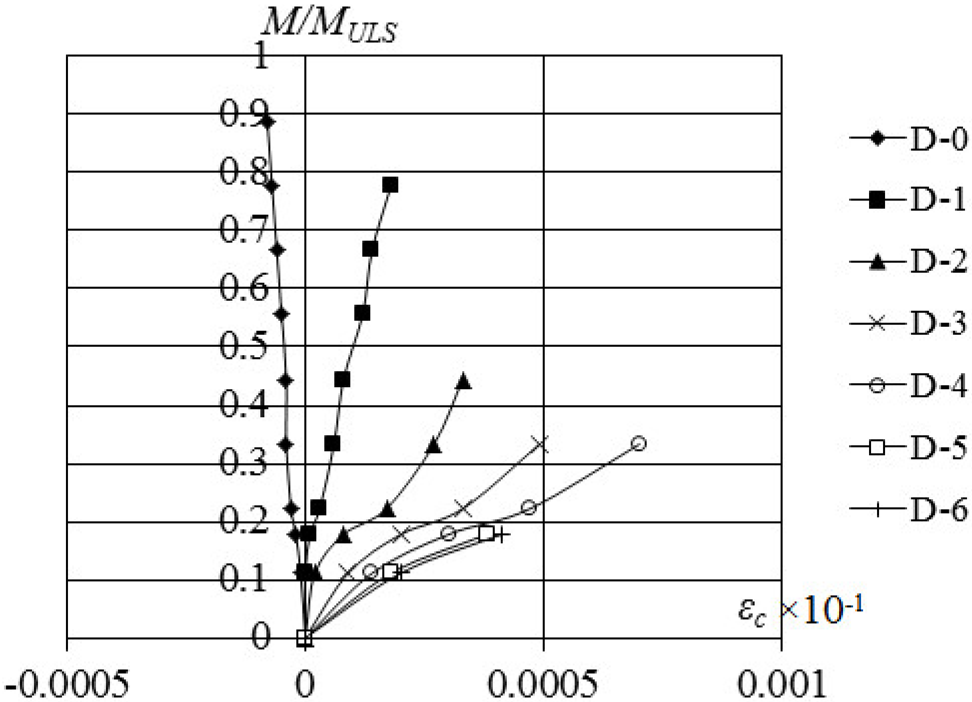


Fig. 3.33.The distribution of longitudinal strains in a beam B13 by the height of the sample (all gauges in the edge).

1. At the edge of the beam the tensile deformations are mainly dominated. Moreover, the sharp increase in deformation with subsequent

breakage of the diagrams in the graphs (Fig.3.34, 3.35, 3.36) shows of the crack formation and further development on the gauge and, consequently, break of the latter (gauge 8 Fig. 3.34; gauge 10 Fig. 3.35).The location of the resistance strain gauges in the sample B 1, see Fig. 2.17.

2. In addition to increasing tensile strain at the edge of the beam along its length, the wave-like character of development of deformations is observed, that is, the transition of deformations in one and the same point of the beam from tension to compression and back (gauges 0, 1, 2 Fig. 3.34; gauge 7 Fig. 3.36). This is due to the formation of cracks extending in close proximity to the gauge, and surrounding it on both sides, because of what short-term compression the location of the gauge happens, causing jumps of strains on the graphs from stretched to a compressed zone.

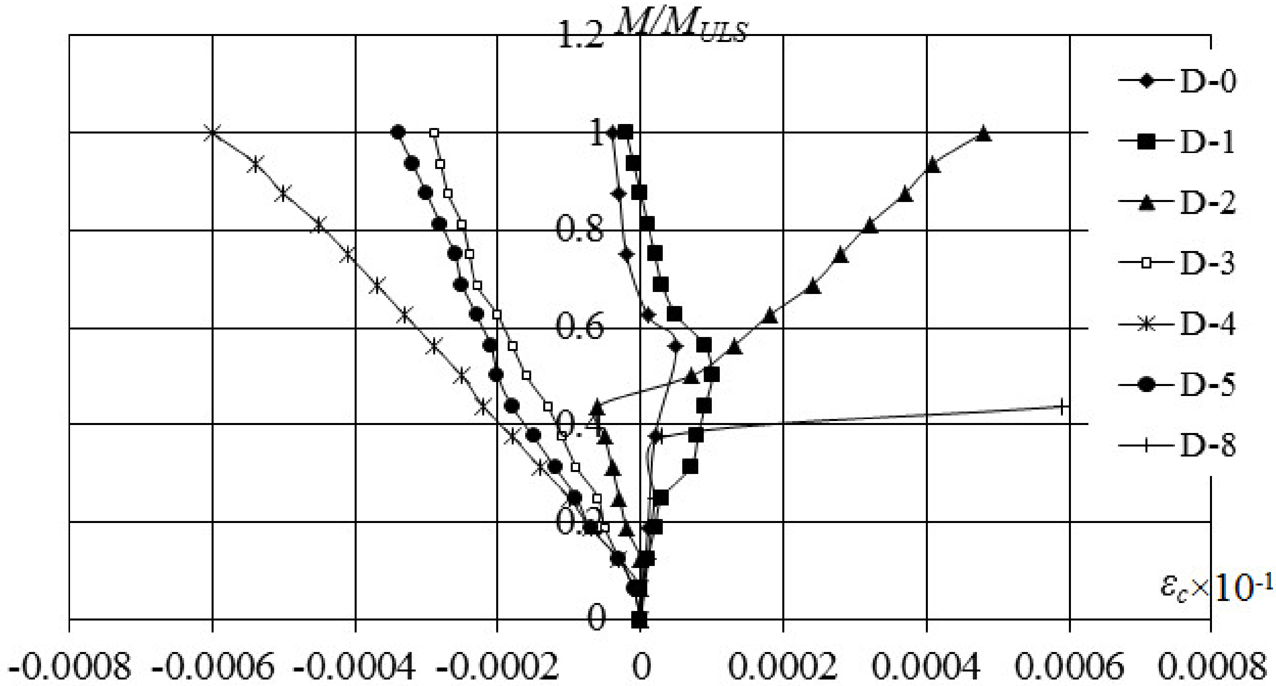


Fig. 3.34.The distribution of longitudinal strains in a beam B 1 on the length of the sample (gauges 0, 1, 2, 8 – edge; 3, 4, 5 – shelf).

3. Shelf of the beams, as mentioned earlier, is in a compressed state throughout the test process, as evidenced by the readings of gauges 3, 4, 5 of beam B 1 (Fig. 3.34) and the gauge 14 (Fig. 3.35).

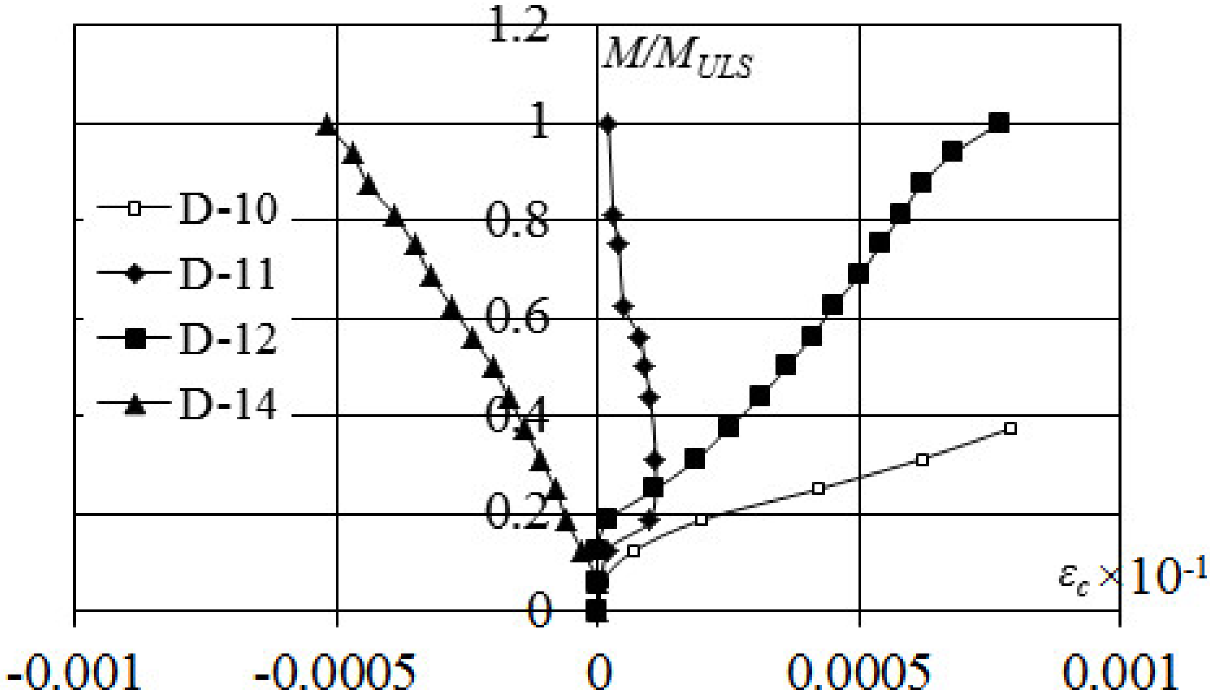


Fig. 3.35.The distribution of longitudinal strains in a beam B 1 by the height of the sample (gauges 10, 11, 12 – edge; 14 –shelf).

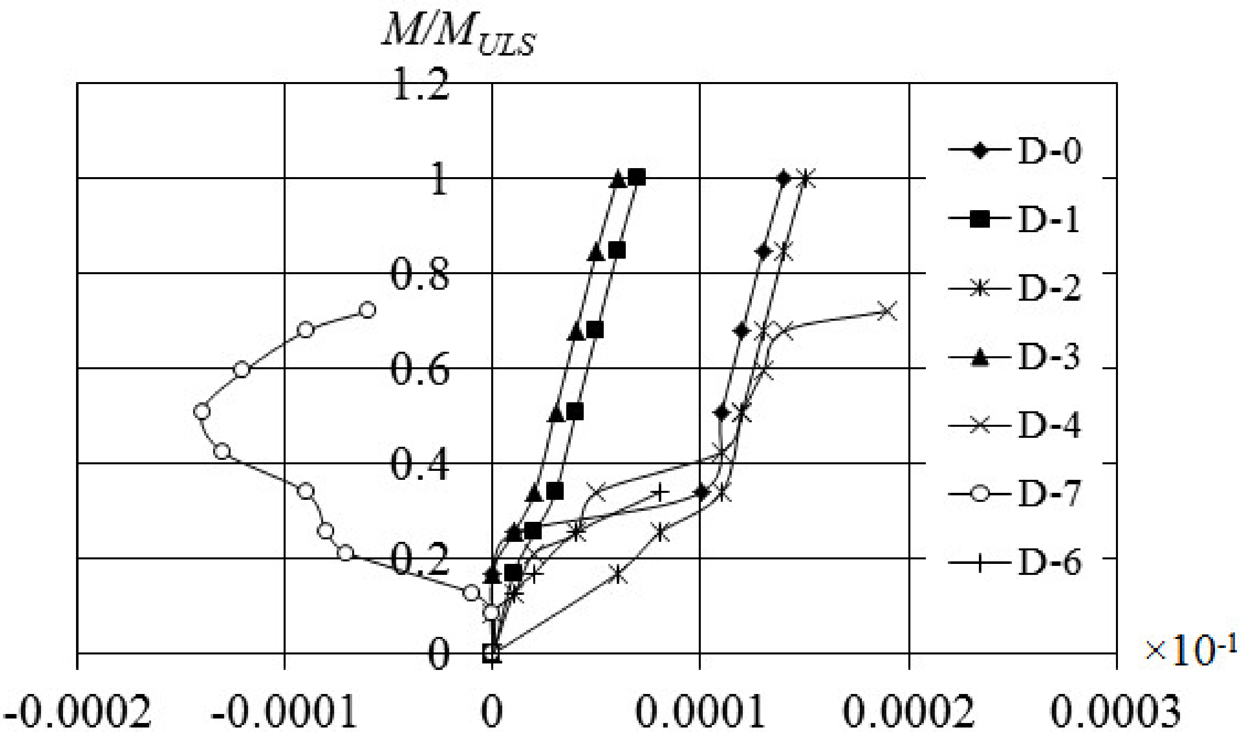


Fig. 3.36.The distribution of longitudinal strains in the beam B 12 on the length of the sample.

3.2.2. The analysis of the development of deformations in the reinforcement. The reinforcement for the duration of the test is stretching. When the load increases the deformations in the reinforcement are rapidly growing (Fig.3.37... 3.39).

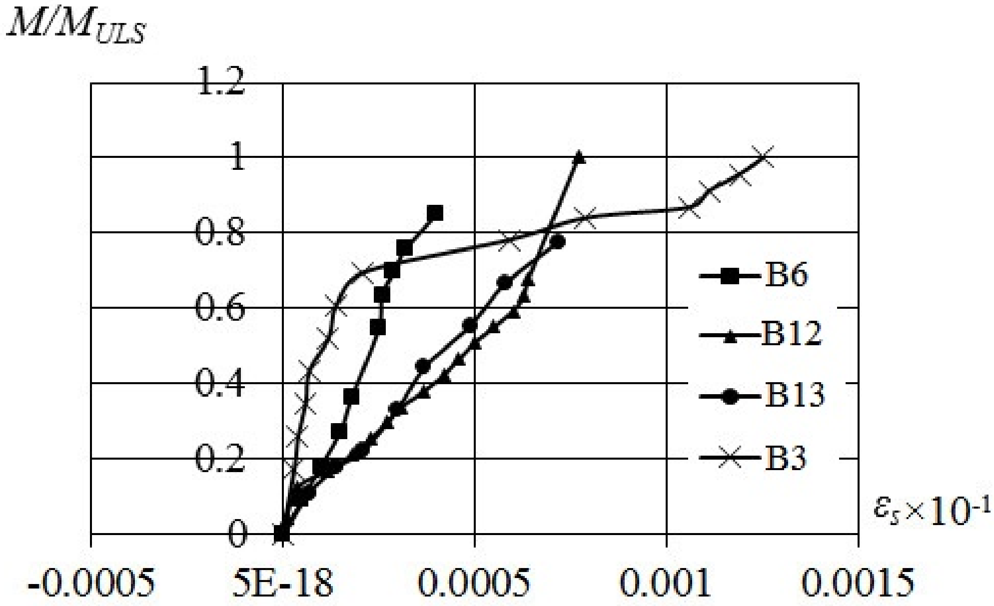


Fig. 3.37.The distribution of deformations in the reinforcement of beams of series 1.

As it can be seen from Fig.3.37 – deformations in the reinforcement of the samples of the first series to the load level (0.7...0.8) M_{ULS} had linear nature of development. At a higher load the increase in deformations of the tension in reinforcement is rapid, it is especially observed in sample B 13, which indicates the development of plastic deformations.

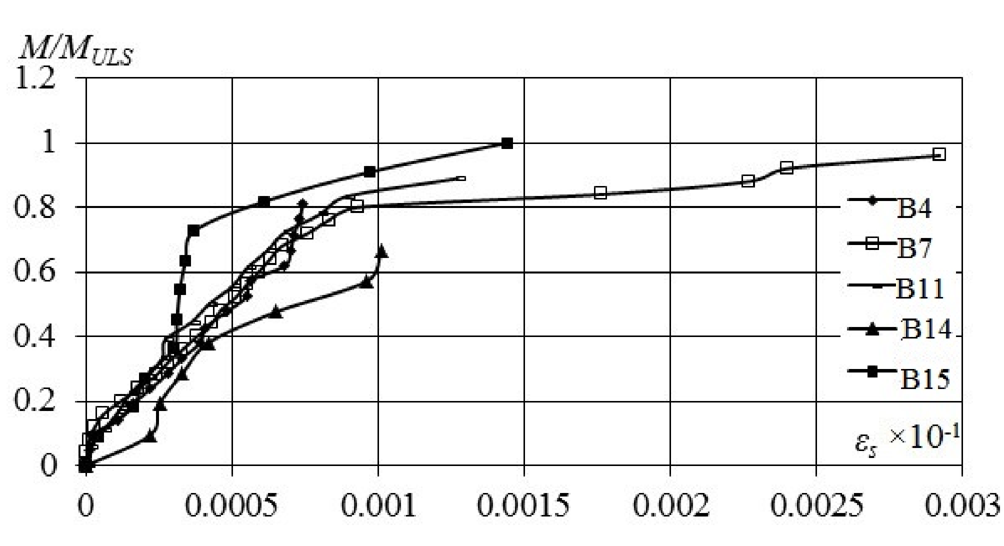


Fig. 3.38.The distribution of deformations in the reinforcement of beams of series 2.

By analyzing the development of deformations in the reinforcement of samples of series 2 it can be said that until the load level (0.6 ... 0.8) M_{ULS} dependence "load - deformation" is linear, which means validity of Hooke's law in this area of the diagram.

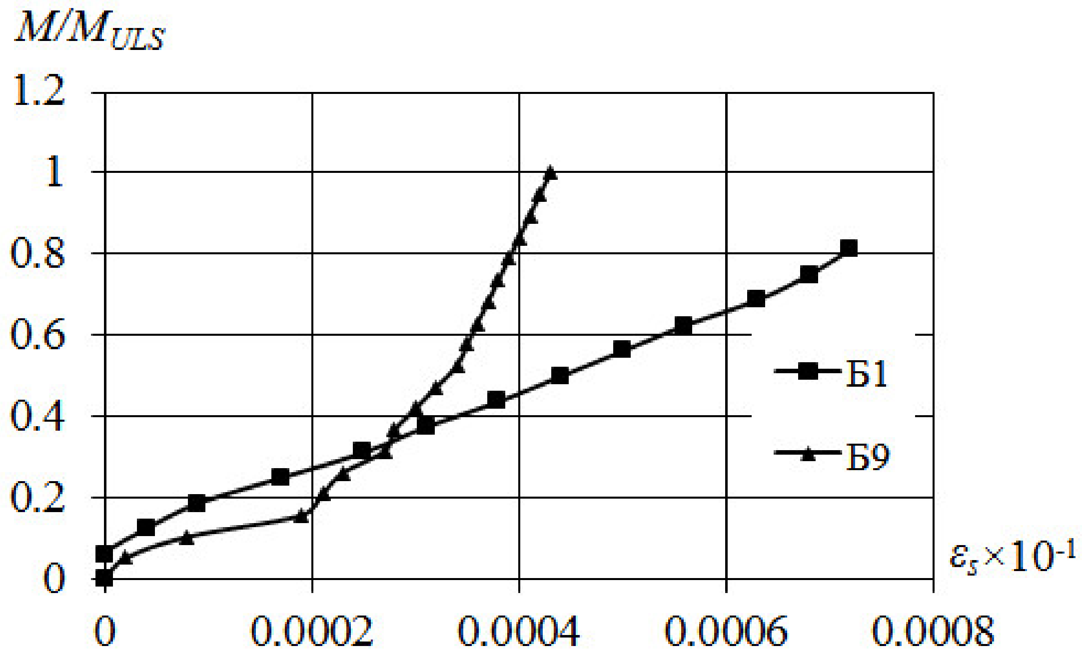


Fig. 3.39.The distribution of deformations in the reinforcement of beams of series 3.

A distinctive feature of the deformation of the reinforcement of the third series of samples is almost linear dependence of *"M/M_{ULS}-□$_s$"*throughout the process of loading of the beam, which indicates the elastic nature of the deformation of the reinforcement.

3.2.3. The analysis of the development of deformations in the reinforcement. As it can be seen from the diagrams of deformation of the reinforcement (Fig.3.37...3.39) on the area 0…(0.6; 0.8; 1) M_{ULS} the elastic deformations dominate, i.e. for determining the stresses in these areas the Hooke's law is valid:

$$\sigma_s = E_s \cdot \varepsilon_s. \qquad (3.2)$$

The nature of the distribution of stresses in the reinforcement of samples of series is shown in Fig. 3.40 ... 3.42.

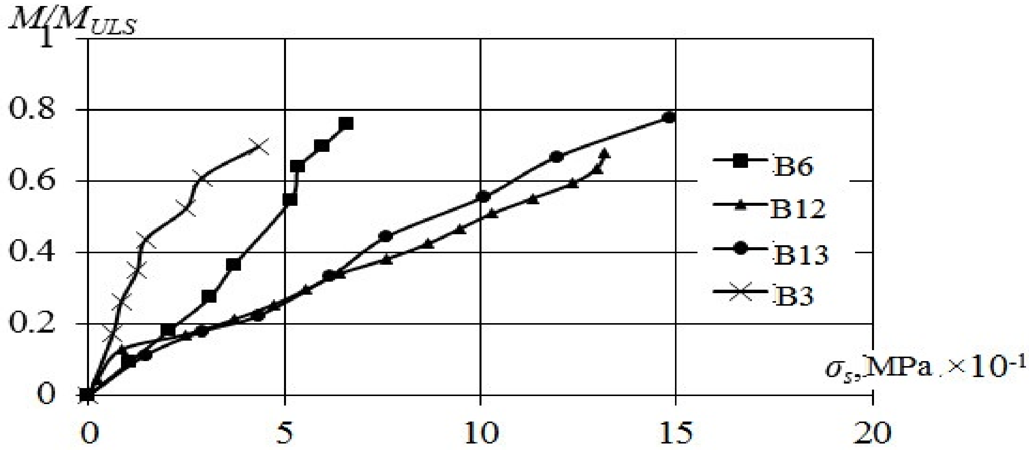


Fig. 3.40.The stress distribution in the reinforcement of series 1.

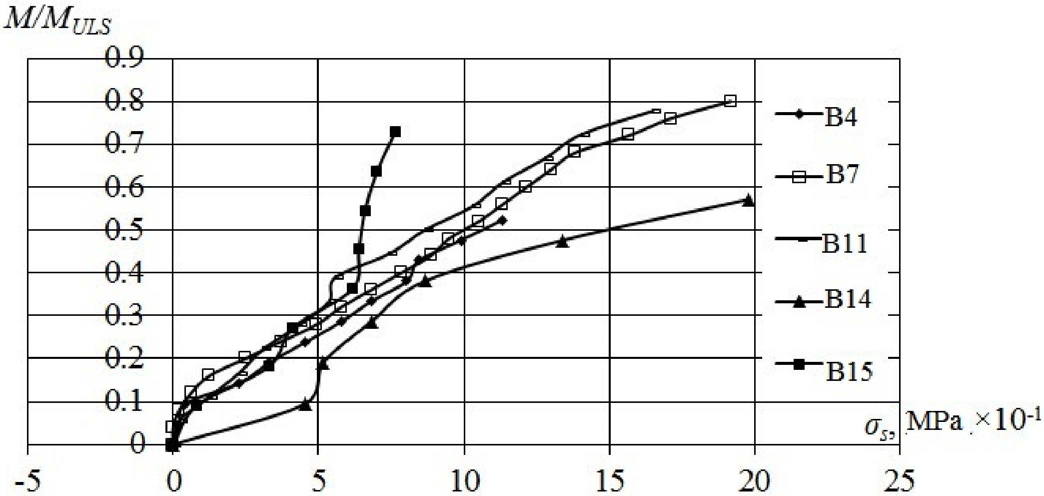


Fig. 3.41 The stress distribution in the reinforcement of series 2.

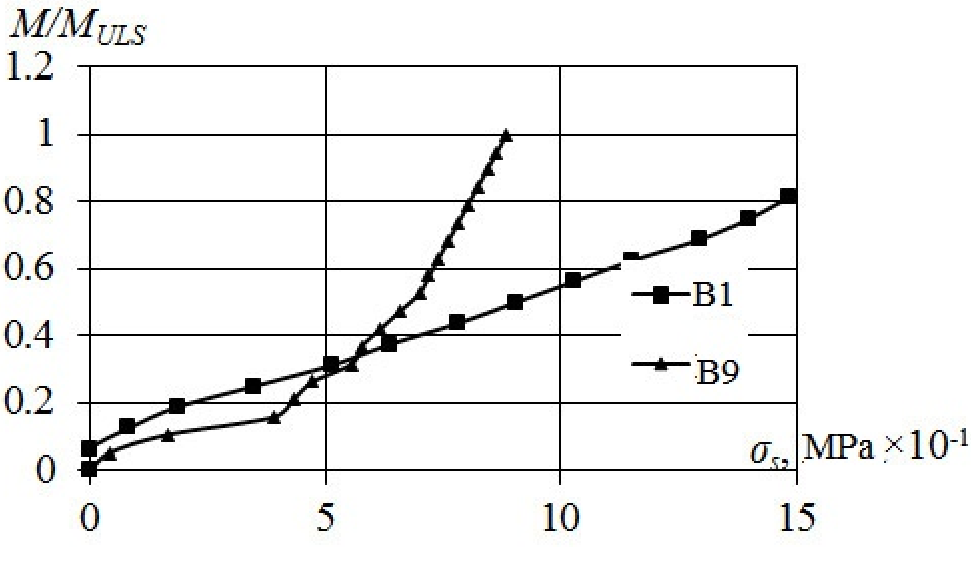


Fig. 3.42 The stress distribution in the reinforcement of series 3.

3.3. Conclusions for the Chapter

1. Experimentally obtained dependence “strain – deformation” during testing of concrete prisms confirms the validity of using non-linear deformation diagram.

2. The actual data obtained on strength of concrete and reinforcement used in the manufacture of samples of beams. The adopted concrete is class C 30/35, reinforcement A 500 C.

3. The conducted tests allowed us to make conclusions about the nature of the destruction of the samples. All the beams collapsed from crushing of concrete at the compressed zone at the section of action of maximum bending moment. The samples for which there has been significant damage to the concrete it was going before yield of a working reinforcement (i.e. samples were repeatedly reinforced), the beams with small damages - there was yield of steel (items not repeatedly reinforced).

4. There were obtained values of the external load corresponding to the appearance of the first normal cracks $F_{w,ult\perp}$ in the zone of pure bending of the samples, and also inclined cracks $F_{w,ult\,/}$ in the spans of the cut of experimental beams and destruction of beams F_{ULS}.

5. The maximum load withstood beams having the least damage (B 3, B 4, B 6, B 7, B 14, B 15), and the lowest - samples with a width of the damaged shelf b_{eff}= 700 mm. A distinctive feature of the deformation of the beams was different nature of development of cracks up to the operational level of loading.

6. From the analysis of diagram obtained using a PC "Compex" it follows that with increase of both the angle of the damage $x_1\{\frac{\beta}{90^0}\}$to 0.5; and the value of damaged part of the shelves $x_3\{\frac{b_{eff1}}{b_{eff2}}\}$to 1 the samples beams are characterized by the lowest residual bearing capacity.Reinforced concrete beam can withstand a maximum external load of 131.5 kN at an angle of damage not more than $15.3^0/90^0$ (X_1=0.17) and the value of damaged parts of the shelves not more than X_3=0.23 (X_3=37.95 mm) in the absence of the depth of damage.

6. By analyzing the obtained values of deflections it is obvious that the greatest bends were received by beams B 7, B 12, B 14, the

distinctive feature of which is the angle of damage $\beta=22.5^0$. The lowest recorded bends in beams of series 3 - B 1, B 2, B 8, the width of the damaged shelf b_{eff} = 700 mm.

7. During the analysis of stress-strain state of the samples – beams there was revealed a number of features:

– the edge of the beam is mainly dominated by tensile strain. With that, the sharp increases in deformation with subsequent breakage of the diagrams on the graphs indicates the formation of cracks and its further development on the gauge and, consequently, rupture of the latter;

– in addition to growing tensile strain in the edge of the beam along its length there was observed wave-like character of development of deformations, which is explained by the formation of cracks extending in close proximity to the gauge and its surrounding on both sides, because of what happens short-term compression the location of the gauge;

– shelf of the beams is in a compressed state throughout the test process.

8. The deformations in the reinforcement mainly to the load level of (0.7 ... 1) M_{ULS} had a linear nature of development. At higher load the increase of the tensile strain in reinforcement goes rapidly, especially observed in sample B 13, which indicates the development of plastic deformation.

CHAPTER 4.

DETERMINATION OF THE RESIDUAL BEARING CAPACITY OF REINFORCED CONCRETE T-BEAMS WITH DAMAGED SHELF

4.1. Modeling of stress-strain state and the determination of residual bearing capacity using the finite element method in PK LIRA – SAPR.

Multifunctional program complex LIRA®-SAPR 2013 is for design and analysis of building structures for various purposes.

For the simulation of NDS and determining destructive torque from the action of external forces there was used one of the latest versions of software, which allows to perform calculations of building structures of various configurations in nonlinear statement that meets the requirements of normative documents.

The calculation is based on the finite element method, using as the primary unknown the displacements and rotations of the nodes of the calculated scheme. In this regard, the idealization of the structures is made in the following form: the system is represented as a set of standard type bodies (rods, plates, shells, etc.), called finite elements and connected to the nodes.

The type of finite element is determined by its geometric form, a physical law describing the relation between internal efforts and internal displacements, and a set of parameters (rigidity) included in the description of this law and others.

A node in the calculation scheme is presented in the form of rigid bodies of infinitesimal size. The node is represented as an object with six degrees of freedom (linear displacements and angles of rotation). All nodes and elements of a design scheme are numbered. The basic system of displacement method is selected by applying at each node of all links. The condition of equality to zero of the effort in these relations is resolving equations of equilibrium and of displacement of said bonds – primary unknown of the displacement method.

The creation of computational models of damaged T-shaped beams was carried out with the software package LIRA-SAPR by constructing a design model from a separate finite elements (KE) (Fig. 4.1, a, b). The concrete was designed with physically non-linear universal spatial isoperimetric volume 6 (KE 234) and 8 (KE 236) nodal elements with dimensions of sides from 5 to 30 mm. KE has a local coordinate system that coincides with the global.

The strength and deformation characteristics of concrete were designed by a piecewise linear law of deformation No. 14, built on the basis of nonlinear diagrams σ-ε with a falling curve [68].

The following parameters were set:– deformation at the intermediate point ε;– the strain at the intermediate point σ.

The number of *i*-points is not limited. If the deformation value obtained in the calculation is beyond the limits of the given law, the exclusion of the material (*EI*=1) of the elementary area of the working section is modeled. The strength characteristics of concrete were set individually for each beam according to the results obtained in the testing of concrete prisms.

Reinforcement carcass (Fig. 4.1, g) was set with physically nonlinear spatial rod finite elements (KE 210). This KE provides the calculation of all types of rod systems with account of physical nonlinearity of the material. The piecewise-linear law of deformation No. 14 was accepted, with a diagram of Prandle and physical yield stress. Strength characteristics of reinforcement were set according to the results of laboratory tests of reinforcing bars.

The total number of finite elements in the design scheme is about 22 thousand nodes – about 24 thousand.

Constraints in the scheme were set as follows (Figure 4.1 *b*): on the left side on a number of nodes the connections were put, limiting the movement on the axes *x, y, z,* on the right - the movement along the *z* axis was limited.

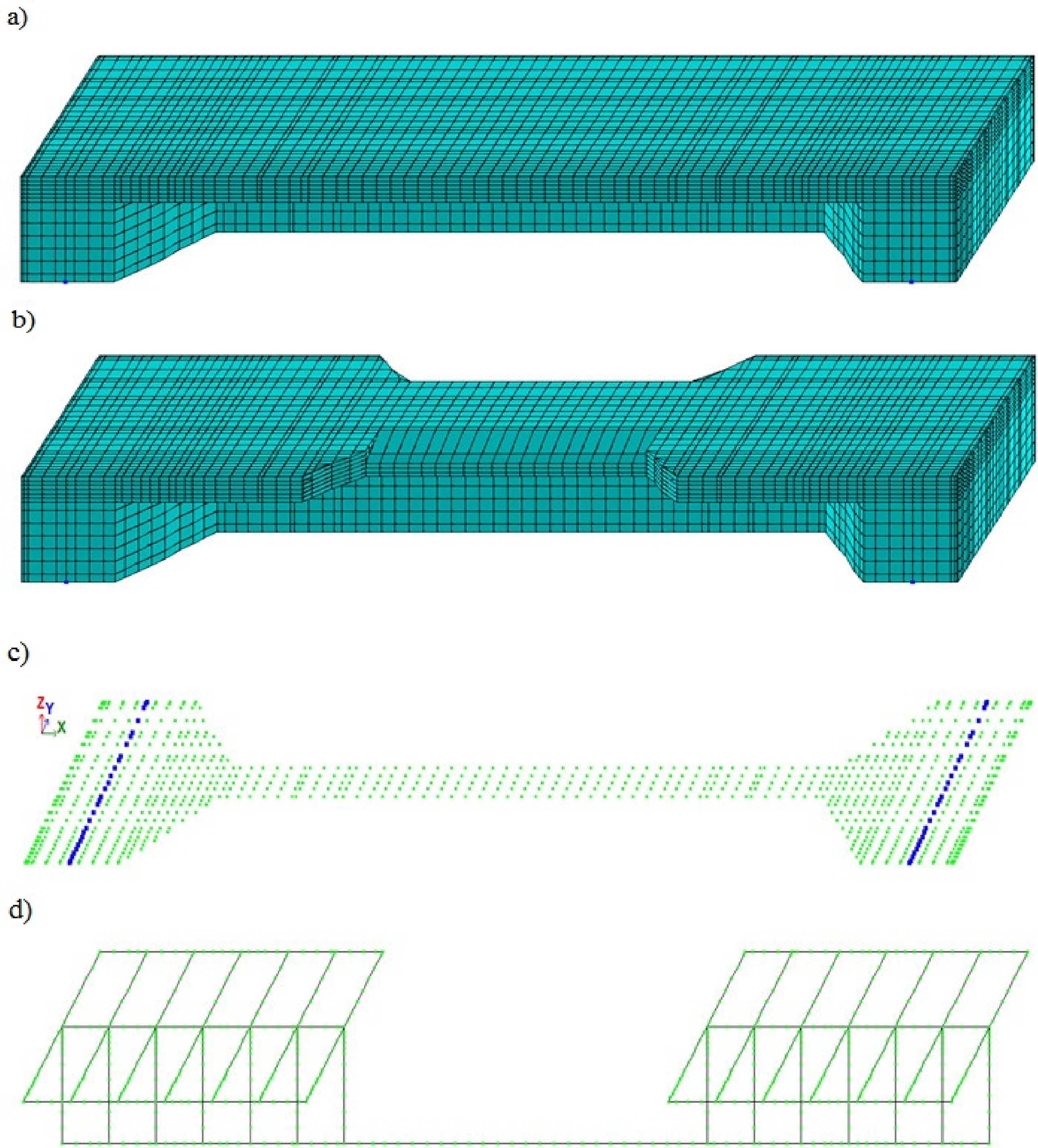


Fig. 4.1. Design scheme B 3: a - a general view of the intact sample, broken into finite elements; b - a model of the damaged T-shaped beam; c - visualization of restraints; d - reinforcement carcass.

The load was transferred directly to the nodes in the thirds of the span. Consideration was also given to the own weight of the construction.

The results of the calculation in the software package are given in table 4.1.The characteristic isofields of stresses for samples in general and in the middle of the span are shown in figures 4.2....4.7.

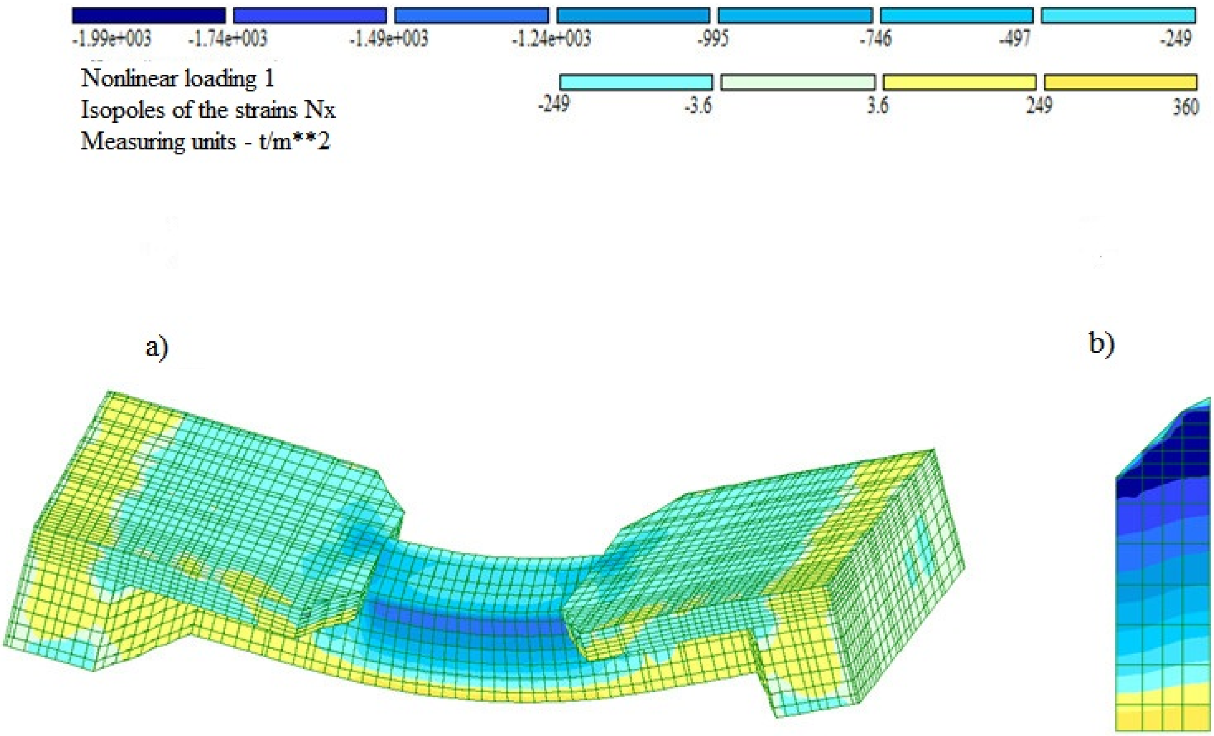


Fig. 4.2. The calculation of the damaged beam B1 in PK LIRA: a – isofields of the strain, general view; b – isofields of the strain in the mid of the span.

When modeling in the software package there were observed some regularities. The destruction of the samples occurred at reaching the maximum stress in the zone of pure bending: for compressed concrete in the upper part in the middle of the span, for stretched – in the lower fibers and in the middle of the span. Also there was a stress concentration in the places of application of force and the injury of the shelf.

There is a similarity with models of destruction during laboratory experiment. So, in the samples with the least damage to the shelves (B 3, B 6, B 12, B 13) the destruction happened after reaching the maximum compressive stress in the shelf. In beams (B 4, B 7, B 10, B 11, B 14, B 15) there was a stress concentration at the place of application of force.

a) b)

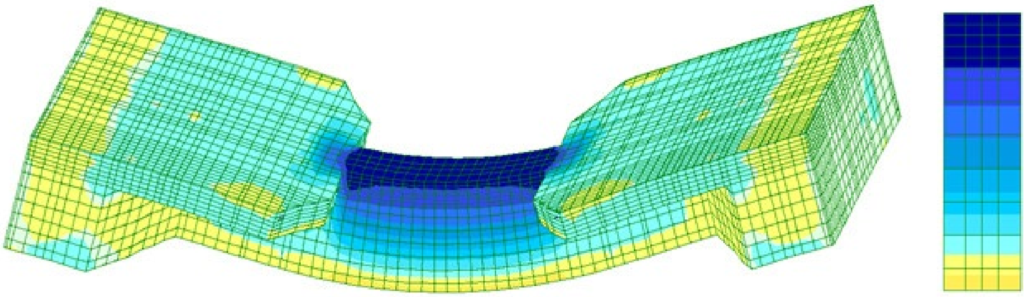


Fig. 4.3. The calculation of the damaged beam B5 in PK LIRA: a – isofields of the strain, general view; b – isofields of the strain in the mid of the span.

a) b)

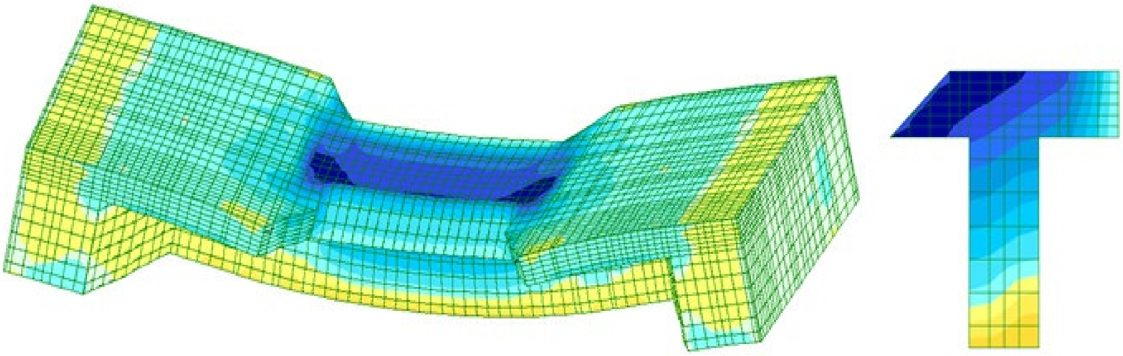


Fig. 4.4. The calculation of the damaged beam B4 in PK LIRA: a – isofields of the strain, general view; b – isofields of the strain in the mid of the span.

The destruction of these samples occurred after reaching the maximum allowable stresses in the zone of maximum bending moment. The most damaged beams B 1, B 2, B 5, B 8, B 9, which in the zone of pure bending had almost no shelves, withstood the smallest load. The most compressed fibers of these beams are in the upper part of the shelf. The destruction of a model also occurs after reaching the maximum stresses in the zone of pure bending.

In the samples that had a damaged shelf at an angle (B 3, B 4, B 7, B 10, B 12) in the middle section there is a rotation of the neutral line (Fig. 4.4, b 4.5, b).

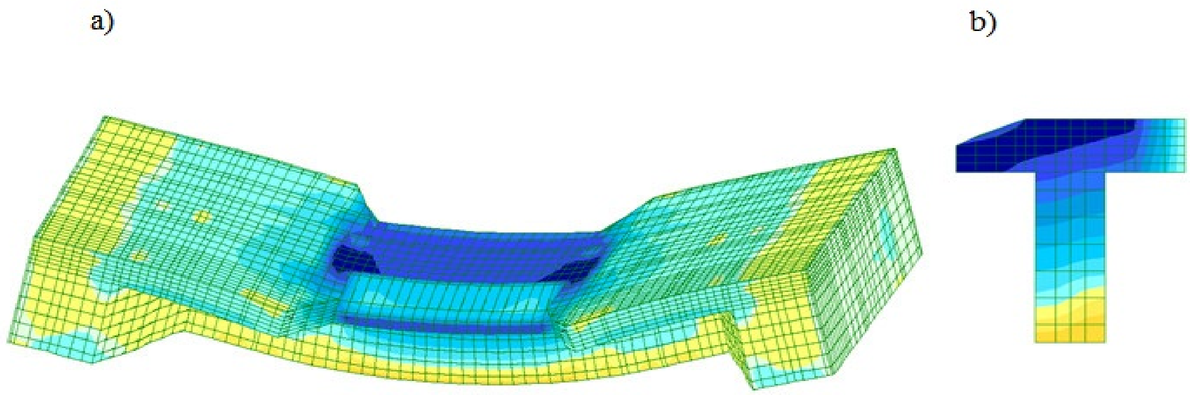


Fig. 4.5. The calculation of the damaged beam B7 in PK LIRA: a – isofields of the strain, general view; b – isofields of the strain in the mid of the span.

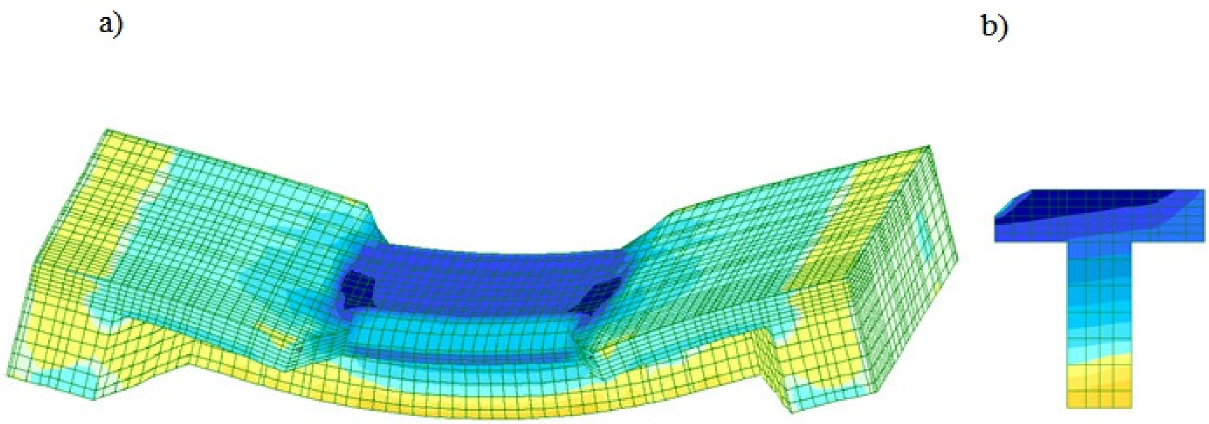


Fig. 4.6. The calculation of the damaged beam B10 in PK LIRA: a – isofields of the strain, general view; b – isofields of the strain in the mid of the span.

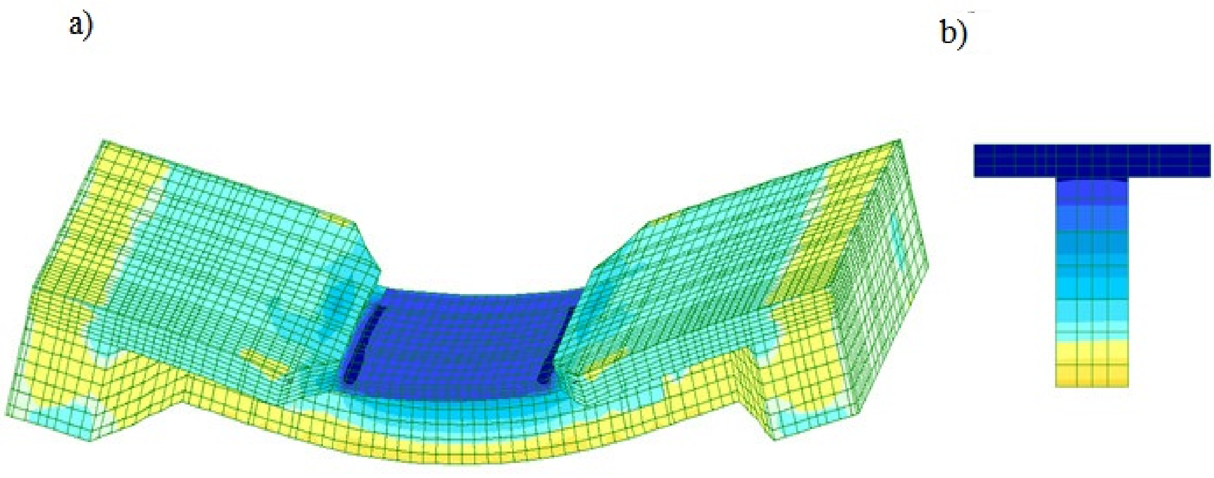


Fig. 4.7. The calculation of the damaged beam B11 in PK LIRA: a – isofields of the strain, general view; b – isofields of the strain in the mid of the span.

Table 4.1

Results of calculation in PK LIRA-SAPR

Label of the beam	F_{ULS}, кN	F_{Lira}, кN	M, kNm	M_{Lira}, kNm	$\frac{M_{Lira}}{M}$
B 1	80	68.67	20	17.17	0.8584
B 2	40	34.34	10	8.58	0.8584
B 3	130	107.91	32.5	26.97	0.8300
B 4	105	93.19	26.25	23.3	0.8875
B 5	95	83.39	23.75	20.85	0.8777
B 6	130	117.72	32.5	29.43	0,9055
B 7	110	93.19	27.5	23.3	0.8472
B 8	75	68.67	18.75	17.17	0.9156
B 9	95	83.39	23.75	20.85	0.8777
B 10	98	96.14	24.5	24.03	0.9810
B 11	90	83.39	22.5	20.85	0.9265
B 12	118	115.76	29.5	28.94	0.9810
B 13	90	81.42	22.5	20.36	0.9047
B 14	105	88.29	26.25	22.07	0.8409
B 15	20	105.95	27.5	26.48	0.9632

The RMS deviation of the results of the calculation:

$$\sigma = \sqrt{\frac{1}{n-1}\sum_{i=1}^{n}(\frac{F_{ULS}}{F_{lira}} - \frac{\overline{F}_{ULS}}{\overline{F}_{lira}})^2} = 0.0598 \approx 6\%;$$

The coefficient of variation:

$$v = \frac{\sigma}{\overline{F}_{ULS}/\overline{F}_{lira}} = 0.054.$$

In general, similar modeling is applicable in the calculation of the damaged beams as it shows a fairly good correlation with results of laboratory experiment (error about 6%) and visually displays the stress-strain state of the investigated samples, but is time-consuming from the point of view of designing and further iterative calculation to determine and clarify the bearing capacity of samples. Therefore, the task of creating a method for calculating the damaged T-shaped beams is relevant.

4.2. Theoretical study of the residual bearing capacity of damaged beams. Case of uniform distribution of compressive efforts.

4.2.1. Background of the calculation. According to the existing rules [68] it is recommended to determine bearing capacity of reinforced concrete bending elements with account of physical nonlinearity of concrete work, i.e. given the real diagram of deformation of concrete ε–σ. Introduction of this diagram will more accurately describe the stress-strain of damaged reinforced concrete T-shaped beams, and thus more accurately assess their residual bearing capacity.

As indicated in section 3.1.7.2 [68] when performing confirmatory calculations of the rectangular cross-sections (or close to this outline) a uniform distribution of normal compressive stresses in the compressed zone is allowed (Fig.4.8). Thus the factor which determines the estimated height of the compressed zone of concrete is taken equal to λ=0.8. Various factors that affect the strength of concrete, taken into account by a factor η, the value of which is recommended to be equal to: η=1,0.

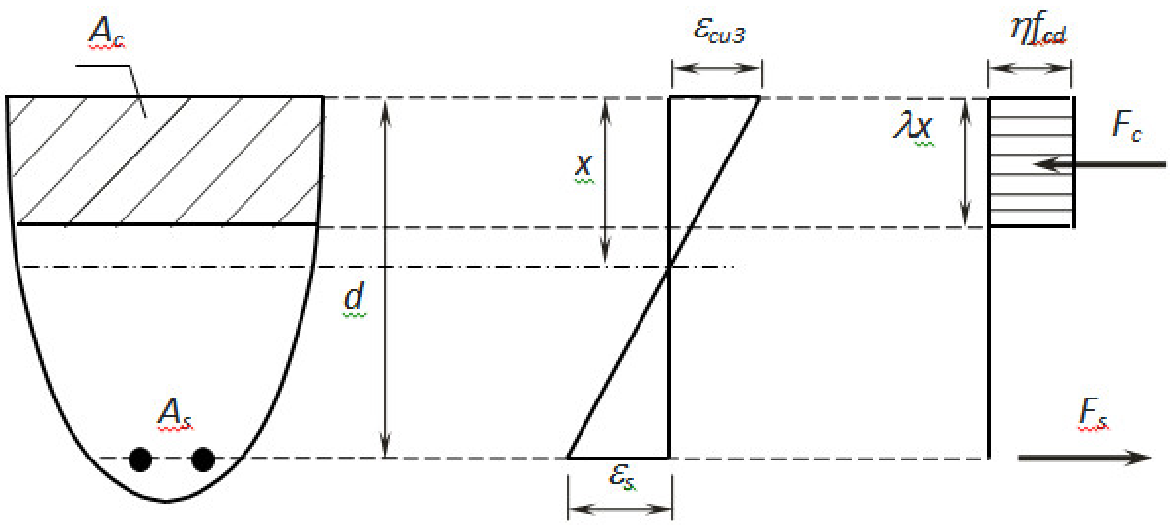


Fig. 4.8.The uniform nature of the stress distribution in the compressed zone.

Since the monograph is developing a methodology for determining the residual bearing capacity of damaged reinforced concrete T-shaped beams that were damaged in the process of operation, this calculation can be considered as calibration and application of the above assumption of uniform stress distribution over the area of the compressed zone should be considered reasonable and not contrary to the requirements of applicable building norms.

When determining the residual bearing capacity of reinforced concrete T-shaped beams the following assumptions are taken into account:

1. The hypothesis of flat cross-sections is accepted, i.e., after deformation the cross sections remain planar and by height of cross-section the deformations change in a linear relation.

2. The stresses in the compressed zone are uniformly distributed with intensity ηf_{cd}.

3. The force in the compressed concrete is applied in the center of mass of the compressed zone of concrete.

4. Work of the stretched concrete in the perception of external forces is not taken into account.

5. Efforts in the tensile zone are fully perceived by reinforcement. The tensile stresses in the reinforcement are not more than the calculated tensile strengths f_t.

6. Front of the damage has a rectilinear shape.

7. The plane of action of external moments and internal forces coincide or are parallel.

Justification of the first assumption is given above. The first, third, fourth and fifth assumptions correspond to the assumptions of the calculating of reinforced concrete bent elements according to current regulations [68]. The sixth assumption is substantiated by the results of field surveys of damaged items as the most common type of destruction. The seventh assumption [12], p. 3.28* is not contrary to applicable building codes and actually means that the point of application of the external longitudinal forces, the resultant of compression forces in the concrete and the reinforcement and the resultant effort in the tensile reinforcement should lie on a straight line.

4.2.2. Design cases. In this work, all studied experimental samples are divided into two groups: damaged T-shaped beams with flat and oblique shelf damage.

The flat will called the damage when its front is parallel to one of the principal axes of the cross section, i.e. the angle of the damage is zero (Fig. 4.9, a). In the case where damage is not parallel to any of the principal axes of the cross section, i.e. the angle of inclination of the front line of damage is not equal to zero, the damage will be considered oblique (Fig. 4.9, b).

The denotation of geometric characteristics of the cross section is the same as in experimental design (see Chapter 2). The height of the compressed zone of concrete is traditionally denoted as *x*.

For finding the residual bearing capacity of mechanically damaged T-shaped beams it is necessary to make the equations of balance.

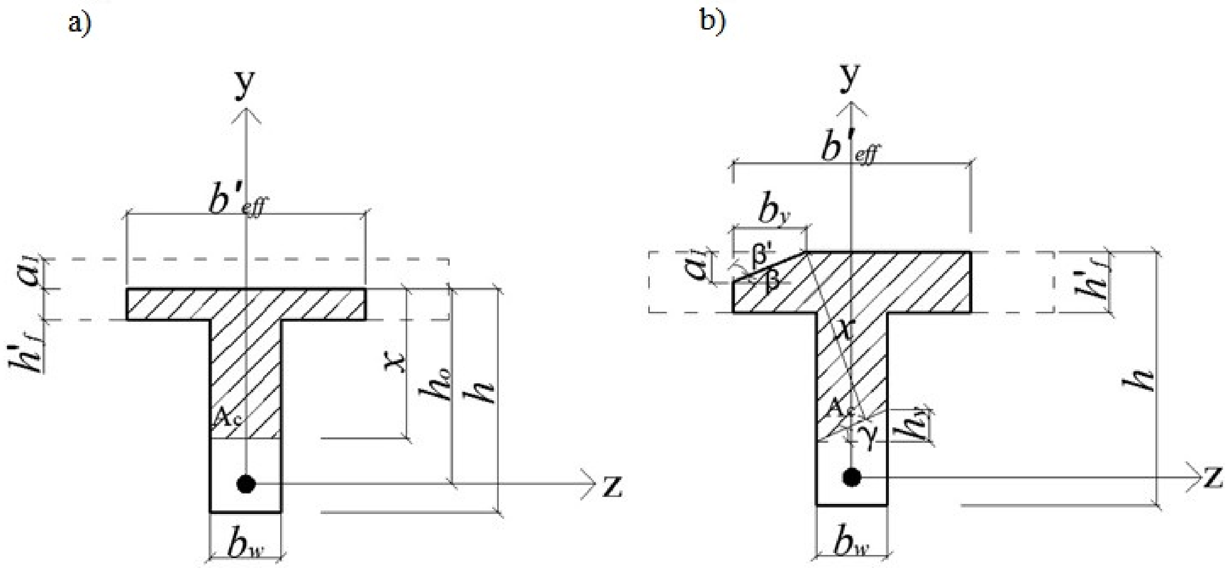


Fig. 4.9. Design cases: a – case of flat damage of the shelf; b – case of oblique damage of the shelf.

When solving this task in the case of flat damage are two unknowns – this is the height of compressed zone of concrete *x* and a critical point *M*. In the case of an oblique damage we have three unknowns – the height of the compressed zone of concrete *x*, the angle of inclination of the neutral line γ and the critical moment *M*.

The considered tasks for the design cases can be resolved in various ways. The Newton method was used in this work for finding the unknown, the algorithm of solving the system in Microsoft Office Excel was created. For the same mathematical transformation the software package MATLAB was used, which allows to reduce the complexity of calculation.

In any of the design cases it is necessary to take into account recommendations [88] with respect to the width of the overhang shelf: the value of b'_{eff}, introduced into the calculation, taking from the condition that the width of the overhang of the shelf in each side of the edge should be no more than 1/6 of the span element and at the cantilever overhangs of the shelf no more than:

1) $h'_f \geq 0,1h$ –take for $6h'_f$;

2) $0,05h \leq h'_f < 0,1h$ – take for $3\ h'_f$;

3) $h'_f < 0,05h$ – overhangs are not counted.

In the general case, the calculation of cross-sections with a shelf in the compressed zone must be carried out depending on the position of the boundary of the compressed zone, i.e., the neutral line is held either in the shelf or in the edge [12]. For T-sections with single reinforcement in the edge:

a). If the boundary of the compressed zone is passed the shelf, the condition must be complied:

$$f_{yd} \cdot A_s \leq f_{cd} \cdot b'_{eff} \cdot h'_f; \tag{4.1}$$

In this case, the area of the compressed zone and the coordinate of the center of gravity is defined as for rectangular section.

b). If the condition (4.1) is not complied, the compressed zone boundary runs in the edge. The calculation must be made with the conditions:

$$M \leq f_{cd} \cdot b_w \cdot x \cdot (h_0 - 0.5 \cdot x) + f_{cd} \cdot \left(b'_{eff} - b_w\right) \cdot h'_f \cdot \left(h_0 - 0.5 \cdot h'_f\right). \tag{4.2}$$

The case of flat damage. Define the position of compressed zone boundary (shelf or edge). For finding the unknown quantities we set up a system of equations.

1. Equation of equilibrium about the axis x (Fig. 4.9, a):

$$f_{cd} \cdot A_c + \sigma_s \cdot A_s = 0; \tag{4.3}$$

2. The equation of the sum of the moments about the x-axis:

$$M - f_{cd} \cdot A_c \cdot y_c = 0. \tag{4.4}$$

In the formulas:

$f_{cd}$is the design value of the concrete strength in compression;

A_s isthe area of the reinforcement;

A_c isthe area of the compressed zone of concrete.

The calculation is divided into two cases – when compressed zone boundary runs in the edge and in the shelf. For T-shaped beams with flat damage the area of the compressed zone will be found by the formula:

1). The boundary of the compressed zone in the shelf area as the rectangular cross section (Fig. 4.10, a):

$$A_c = b'_{eff} \cdot x; \tag{4.5}$$

2). The boundary of the compressed zone at the edge (Fig. 4.10, b):

$$A_c = b'_{eff} \cdot h'_f + b_w \cdot (x - h'_f). \tag{4.6}$$

The stress σ_s in the tensile reinforcement should be determined according to the known empirical formula:

$$\sigma_{si} = \frac{\sigma_{sc,u}}{1-\frac{\omega}{1,1}}\left(\frac{\omega}{\xi_i} - 1\right), \tag{4.7}$$

where

$$\xi_i = \frac{x}{h_{oi}}, \tag{4.8}$$

$h_{oi}$is the distance from the axis passing through the center of gravity of the cross section of the considered i-th rod, and parallel line, limiting the compressed area, to the most remote point of the compressed zone of the cross section.

In the case of flat damage this value is easily determined:

$$h_0 = h - \left(a + \frac{d}{2}\right), \tag{4.9}$$

where$a$ is the protective layer of concrete;

d is the diameter of the reinforcement.

In the formula (4.7):

ωis the characteristic of compressed zone of concrete determined by the formula [88]:

$$\omega = \alpha - 0.008 \cdot f_{cd}, \tag{4.10}$$

For heavy concrete coefficient α is 0.85.

In equation (4.4):y_c is the coordinate of the center of gravity of the compressed concrete (Fig.4.10, a, b).

1). The boundary of the compressed zone in the shelf:

$$y_c = h_0 - \frac{x}{2}; \quad (4.11)$$

2). The boundary of the compressed zone at the edge:

$$y_c = \frac{A_1 \cdot y_1 + A_2 \cdot y_2}{A_1 + A_2}, \tag{4.12}$$

where:

$$A_1 = b'_{eff} \cdot h'_f; \quad (4.13)$$

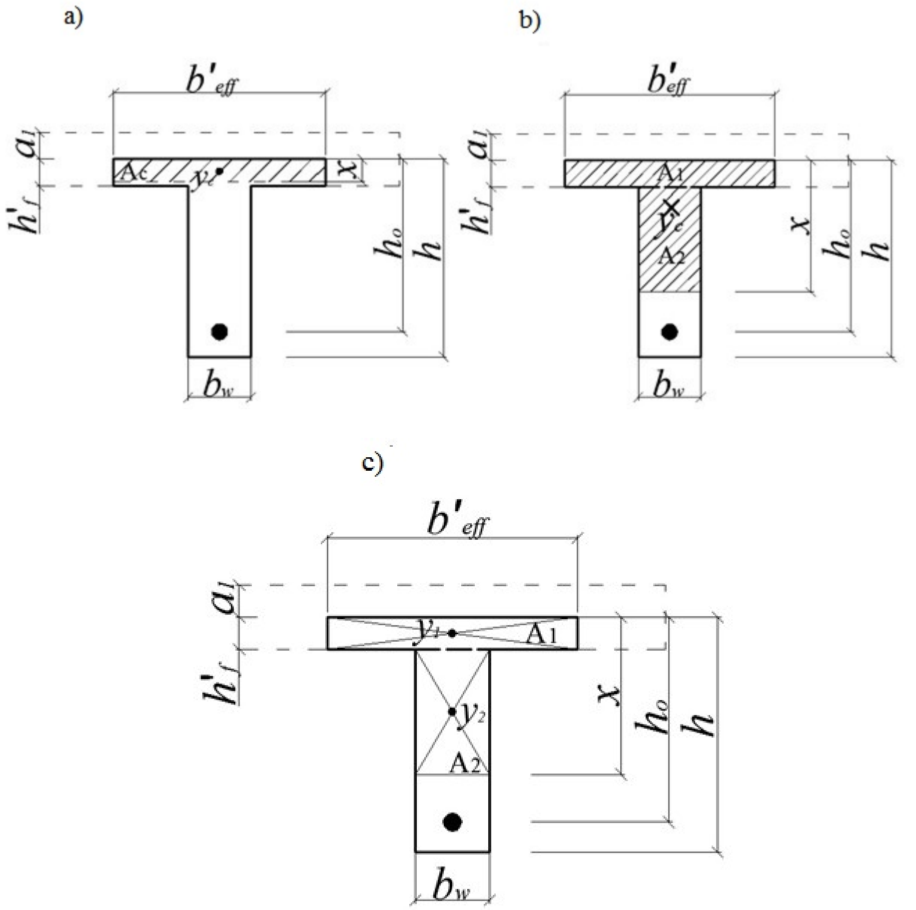


Fig. 4.10. To determine the coordinates of the center of gravity of the compressed concrete at the flat damage: a - the point of application of the resultant of the compressed concrete y_c, the boundary of the compressed zone in the shelf; b – the point of application of the resultant of the compressed concrete y_c, the boundary of the compressed zone at the edge; c – the division of a compressed zone on simple shapes.

$$A_2 = b_w \cdot (x - h'_f); \tag{4.14}$$

$$y_1 = h_0 - \frac{h'_f}{2}; \tag{4.15}$$

$$y_2 = h_0 - x + \frac{(x - h'_f)}{2}. \tag{4.16}$$

Thus, solving the system of equations (4.3, 4.4), we obtain the value of the unknown quantities *x* and *M*. It is necessary to consider that the value of the desired parameters are positive and cannot be less than zero, therefore, of obtained roots of the equations select the appropriate.

The known the height of the compressed zone allows determining the tensile stress in the reinforcing bar according to the formula (4.7). Tensile stresses should not exceed the limit stress $\sigma_{si} \leq f_{yd}$. If this condition is not met, then accept that $\sigma_{si} = f_{yd}$, and make a recount considering this in mind.

Case of an oblique injury. For finding the unknown variables *x*, γ and *M* it is necessary to form a system of three equations:
1. Equation of equilibrium about the axis *x* (Fig.4.9, a):

$$f_{cd} \cdot A_c + \sigma_s \cdot A_s = 0; \tag{4.17}$$

2. The equation of the sum of the moments about the axis *x*:

$$M - f_{cd} \cdot A_c \cdot y_c = 0; \tag{4.18}$$

3. The static moment of the compressed zone of the concrete relative to the axis y (Fig.4.13):

$$S_y = A_1 x_1 - A_2 x_2. \tag{4.19}$$

In equation (4.19) instead of the equation of equilibrium of moments the static moment is considered, as the strains are uniform over the area.

For a T-shaped beam with an oblique damage the area of the compressed zone is determined based on the section geometry and parameters of the damage by summing the areas of simple figures, on which the compressed area is splitted (Fig. 4.11).

To determine the area of the compressed zone at an oblique damage we conditionally set the position of the neutral line, determine the required value for composing a system of equations and make the calculation.In that case, if the system (4.19 ... 4.17) has no roots or that they are negative, it means that the shape of the compressed zone selected is not true, therefore, it is necessary to consider another position of compressed zone boundary. In general, we can distinguish two design cases. The first case – when the boundary of the compressed zone runs in the edge and the second case is when it is in the shelf. Let us consider these cases separately.

1). The boundary of the compressed zone at the edge. We assume that the neutral line passes as shown in Fig. 4.11, a. Then in this case:

$$A_c = b'_{eff} \cdot h'_f - \frac{1}{2} \cdot a_1 \cdot b_y + b_w \cdot (x - h'_f) - \frac{1}{2} \cdot b_w \cdot h_y; \quad (4.20)$$

$$b_y = a_1 \cdot tg\beta'; \quad (4.21)$$

$$h_y = b_w \cdot tg\gamma. \quad (4.22)$$

In the case of an oblique damage value h_o (Fig. 4.11, b), which is included in the formula (4.8) we find from the triangle Δ CAD. Angles: < CAD=90°, <AEC=γ.

The required value of h_o is the leg AE of triangle under consideration.

$$h_0 = \mathrm{AE} = EC \cdot \cos\gamma; \quad (4.23)$$

$$EC = ED + DC; \quad (4.24)$$

$$ED = h - \left(a + \frac{d}{2}\right). \quad (4.25)$$

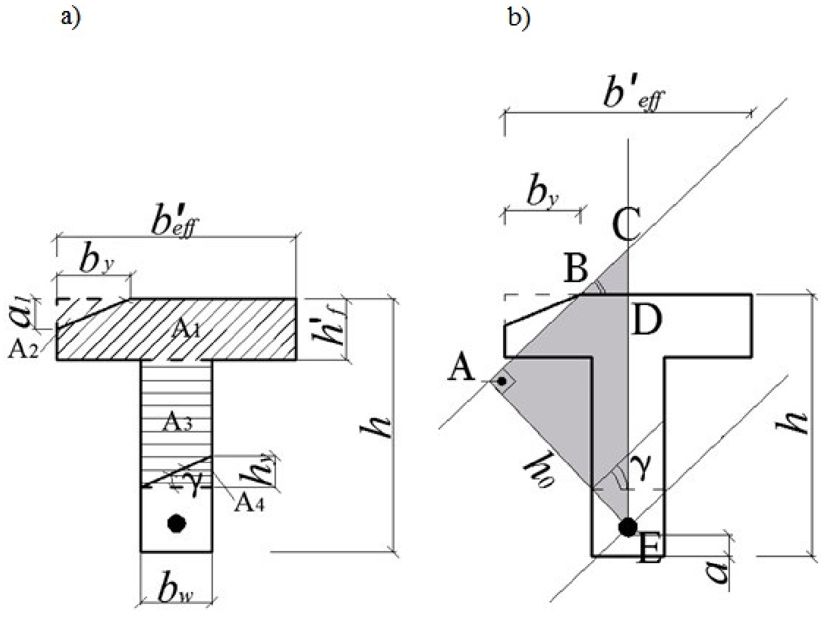


Fig.4.11. Geometric characteristics at an oblique damage: a – division of the section into simple shapes, the boundary of the compressed zone at the edge; b – determination of the h_{oi}.

From the triangle Δ BDC (<BDC = 90 °, <CBD = γ) find DC:

$$DC = BD \cdot \mathrm{tg}\gamma; \quad (4.26)$$

$$BD = \frac{b'_{eff}}{2} - b_y. \tag{4.27}$$

Substituting these expressions in (4.24) we get:

$$h_0 = \left(h - \left(a + \frac{d}{2}\right) + \left(\frac{b'_{eff}}{2} - b_y\right) \cdot \text{tg}\gamma\right) \cdot \cos\gamma. \tag{4.28}$$

Unknown coordinate of the center of gravity of the compressed concrete y_c (Fig.4.12, a):

$$y_c = \frac{A_1 \cdot y_1 - A_2 \cdot y_2 + A_3 \cdot y_3 - A_4 \cdot y_4}{A_1 - A_2 + A_3 - A_4}, \tag{4.29}$$

where(Fig.4.12, b):

$$A_1 = b'_{eff} \cdot h'_f; \tag{4.30}$$

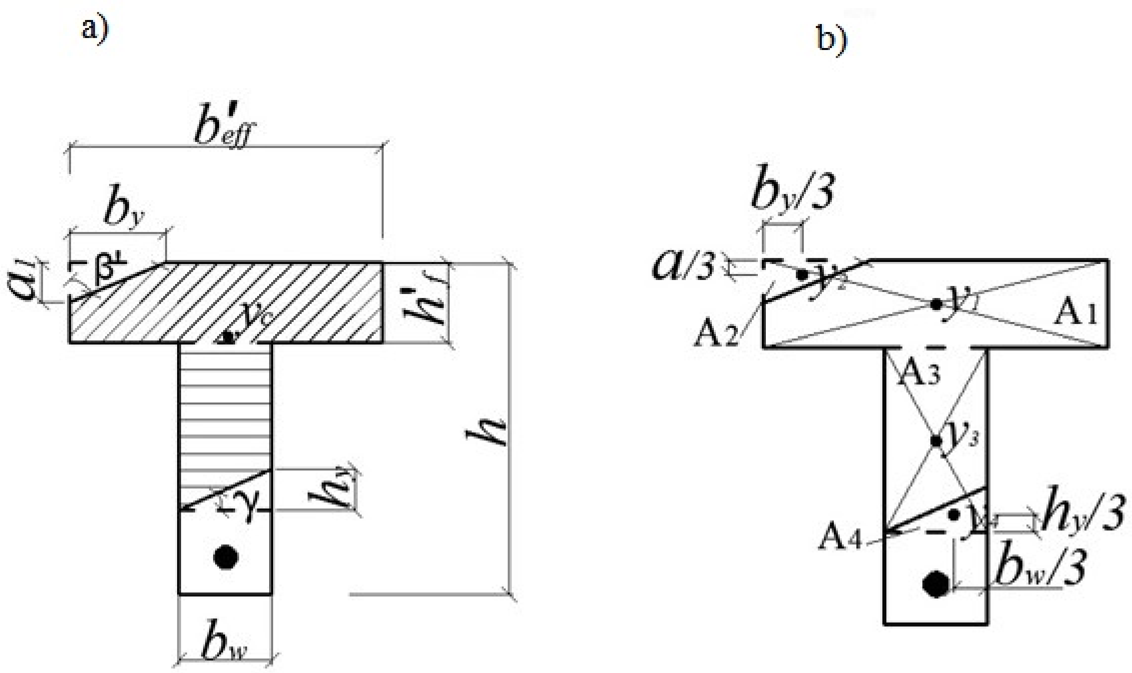


Fig. 4.12. To determine the coordinates of the center of gravity of the compressed concrete at an oblique damage: a - the point of application of the resultant of compressed concrete y_c; b – dividing the compressed areas on simple shapes.

$$A_2 = \frac{1}{2} a_1 \cdot b_y = \frac{1}{2} a_1^2 \cdot tg\beta'; \tag{4.31}$$

$$A_3 = (x - h'_f) \cdot b_w; \tag{4.32}$$

$$A_4 = \frac{1}{2} b_w \cdot h_y = \frac{1}{2} b_w^2 \cdot tg\gamma; \tag{4.33}$$

$$y_1 = h - \left(a + \frac{d}{2}\right) - \frac{h'_f}{2}; \tag{4.34}$$

$$y_2 = h - \left(a + \frac{d}{2}\right) - \frac{a_1}{3}; \tag{4.35}$$

$$y_3 = h - \left(a + \frac{d}{2}\right) - x + \frac{(x - h'_f)}{2}; \tag{4.36}$$

$$y_4 = h - \left(a + \frac{d}{2}\right) - x + \frac{b_w \cdot tg\gamma}{3}. \tag{4.37}$$

In equation (4.19) the static moment of the compressed zone of the concrete relative to the axis *y* shall be defined by means of splitting into simple shapes (Fig.4.13, b):

$$A_1 = A_3 - A_4 + A_5 - A_6; \tag{4.38}$$

$$x_1 = \frac{A_3 \cdot x_3 - A_4 \cdot y_4 + A_5 \cdot y_5 - A_6 \cdot y_6}{A_3 - A_4 + A_5 - A_6}; \tag{4.39}$$

$$A_2 = A_7 + A_8 - A_9; \tag{4.40}$$

$$x_2 = \frac{A_7 \cdot x_7 + A_8 \cdot y_8 - A_9 \cdot y_9}{A_7 + A_8 - A_9}. \tag{4.41}$$

where

$$A_3 = \frac{b'_{eff} - b_w}{2} \cdot h'_f; \tag{4.42}$$

$$A_4 = \frac{1}{2} \cdot b_y \cdot a_1 = \frac{1}{2} \cdot a_1^2 \cdot tg\beta'; \tag{4.43}$$

$$A_5 = x \cdot \frac{b_w}{2}; \tag{4.44}$$

$$A_6 = \frac{1}{2} \cdot \frac{b_w}{2} \cdot \frac{b_w \cdot tg\gamma}{2}; \tag{4.45}$$

$$A_7 = \frac{b'_{eff} - b_w}{2} \cdot h'_f; \tag{4.46}$$

$$A_8 = \left(x - \frac{tg\gamma \cdot b_w}{2}\right) \frac{b_w}{2}; \tag{4.47}$$

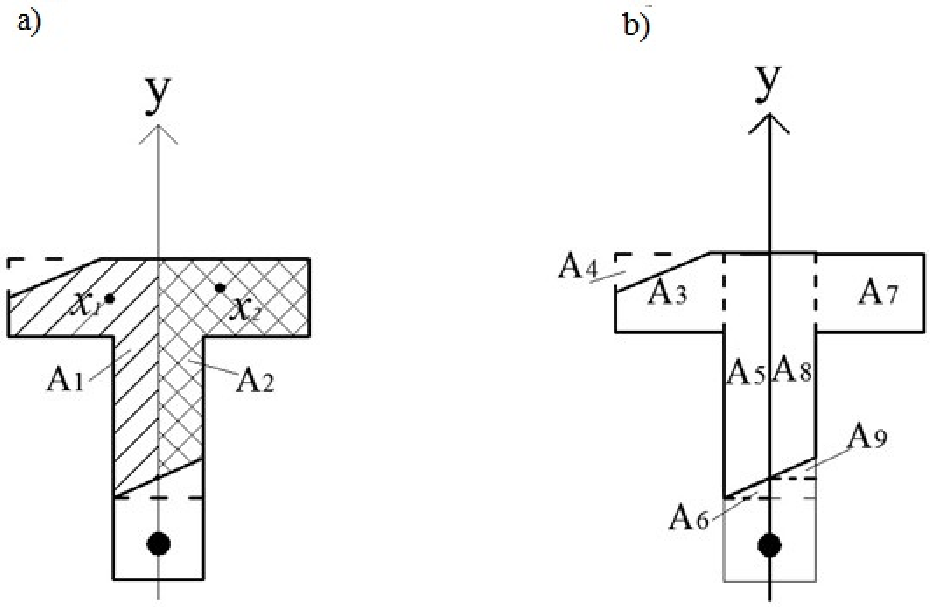


Fig. 4.13. Static moment about main axis y, the boundary of the compressed zone at the edge: a - balance of parts of the cross section about the y axis; b – simple composite figures.

$$A_9 = \frac{1}{2} \cdot \frac{b_w}{2} \cdot \frac{b_w \cdot tg\gamma}{2}; \tag{4.48}$$

$$x_3 = \frac{b'_{eff} - b_w}{4} + \frac{b_w}{2}; \tag{4.49}$$

$$x_4 = \frac{b'_{eff}}{2} - \frac{b_y}{3} = \frac{b'_{eff}}{2} - \frac{a_1 . tg\beta\prime}{3}; \tag{4.50}$$

$$x_5 = \frac{b_w}{4}; \tag{4.51}$$

$$x_6 = \frac{b_w}{6}; \tag{4.52}$$

$$x_7 = \frac{b'_{eff} - b_w}{4} + \frac{b_w}{2}; \tag{4.53}$$

$$x_8 = \frac{b_w}{4}; \tag{4.54}$$

$$x_9 = \frac{b_w}{3}. \tag{4.55}$$

2). For the case where the compressed zone boundary runs in a shelf, we describe the main differences. Value needed to write the system of equations (4.17... 4.19) are defined similarly to the previous case. Suppose that the boundary of the compressed zone is in the shelf as follows (see Fig. 4.14, a), then the area of the compressed zone is:

$$A_c = b'_{eff} \cdot h'_f - \frac{1}{2} \cdot a_1 \cdot b_y - \frac{1}{2} \cdot b'_{eff} \cdot h_y; \quad (4.56)$$

$$h_y = b'_{eff} \cdot tg\gamma. \quad (4.57)$$

Unknown coordinate of the center of gravity of the compressed concrete y_c:

$$y_c = \frac{A_1 \cdot y_1 - A_2 \cdot y_2 - A_3 \cdot y_3}{A_1 - A_2 - A_3}, \quad (4.58)$$

where(Fig.4.14, *b*):

$$A_1 = b'_{eff} \cdot h'_f; \quad (4.59)$$

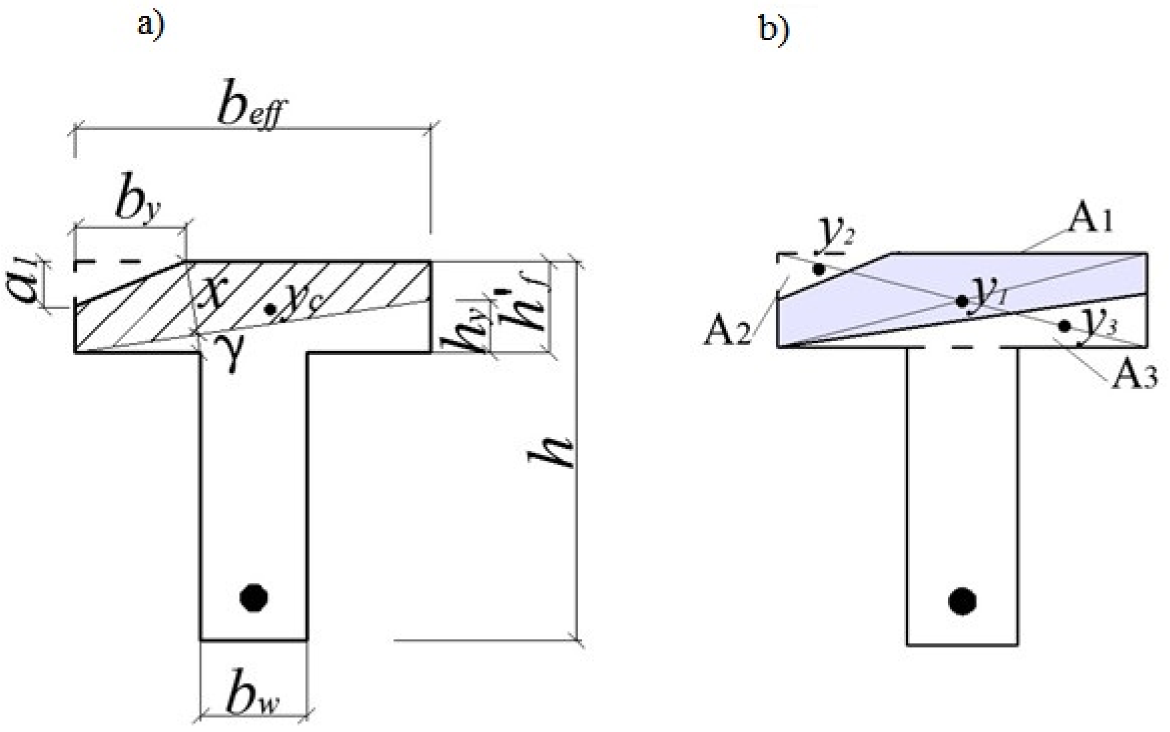


Fig.4.14. For determining the area of the compressed zone: a - the boundary of the compressed zone in the shelf; b – dividing the compressed zones on simple shapes.

$$A_2 = \frac{1}{2} a_1 \cdot b_y = \frac{1}{2} a_1^2 \cdot tg\beta'; \quad (4.60)$$

$$A_3 = \frac{1}{2} b'_{eff} \cdot h_y = \frac{1}{2} b'^{\,2}_{eff} \cdot tg\beta'; \quad (4.61)$$

$$y_1 = h - \left(a + \frac{d}{2}\right) - \frac{h'_f}{2}; \quad (4.62)$$

$$y_2 = h - \left(a + \frac{d}{2}\right) - \frac{a_1}{3}; \quad (4.63)$$

$$y_3 = h - \left(a + \frac{d}{2}\right) - h'_f + \frac{h_y}{3}. \quad (4.64)$$

In equation (4.19) the static moment of the compressed zone of the concrete relative to the axis y (Fig. 4.15):

$$A_1 = A_3 - A_4 - A_5; \quad (4.65)$$

$$x_1 = \frac{A_3 \cdot x_3 - A_4 \cdot y_4 - A_5 \cdot y_5}{A_3 - A_4 - A_5}; \quad (4.66)$$

$$A_2 = A_6 - A_7; \quad (4.67)$$

$$x_2 = \frac{A_6 \cdot y_6 - A_6 \cdot y_6}{A_6 - A_6}. \quad (4.68)$$

where:

$$A_3 = \frac{b'_{eff}}{2} \cdot h'_f; \quad (4.69)$$

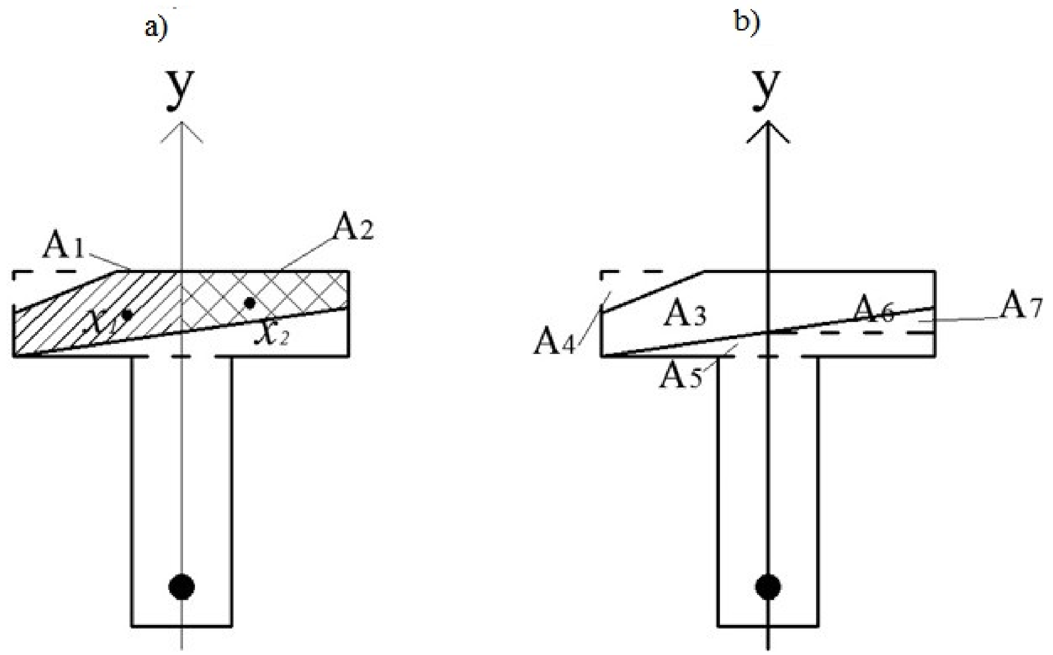


Fig. 4.15. Static moment about main axis y, the boundary of the compressed zone in the shelf: a – balance of parts of the cross section about the y axis; b – simple composite figures.

$$A_4 = \frac{1}{2} a_1 \cdot b_y = \frac{1}{2} a_1^2 \cdot tg\beta'; \qquad (4.70)$$

$$A_5 = \frac{1}{2} \cdot \left(\frac{b'_{eff}}{2}\right)^2 \cdot tg\gamma; \qquad (4.71)$$

$$A_6 = \frac{b'_{eff}}{2} \cdot \left(h'_f - \frac{h_y}{2}\right) = \frac{b'_{eff}}{2} \cdot \left(h'_f - \frac{b'_{eff} \cdot tg\gamma}{2}\right); \qquad (4.72)$$

$$A_7 = \frac{1}{2} \cdot \left(\frac{b'_{eff}}{2}\right)^2 \cdot tg\gamma; \qquad (4.73)$$

$$x_3 = \frac{b'_{eff}}{4}; \qquad (4.74)$$

$$x_4 = \frac{b'_{eff}}{2} - \frac{b_y}{3} = \frac{b'_{eff}}{2} - \frac{a_1 \cdot tg\beta\prime}{3}; \qquad (4.75)$$

$$x_5 = \frac{b'_{eff}}{6}; \qquad (4.76)$$

$$x_6 = \frac{b'_{eff}}{4}; \qquad (4.77)$$

$$x_7 = \frac{b'_{eff}}{2} - \frac{b'_{eff}}{6} = \frac{b'_{eff}}{3}. \qquad (4.78)$$

By solving the system of equations (4.15 4.17...), we get the value of the unknown quantities x, γ, and m. Like in the calculation of tasks with a flat damage it is necessary to remember that x is a positive value, of the found roots choose the suitable. The tensile stresses must not exceed the limit stresses $\sigma_{si} \leq f_{yd}$. If this condition is not met, then we accept that $\sigma_{si} = f_{yd}$, and make a recount considering this in mind.

4.2.3. The results of calculation of damaged T-shaped beams. The results of the calculation by the above method are presented in table 4.2.

The RMS deviation for the calculation results:

$$\sigma = \sqrt{\frac{1}{n-1} \sum_{i=1}^{n} \left(\frac{M}{M_{calc}} - \frac{\overline{M}}{\overline{M}_{calc}}\right)^2} = 0.1464 \approx 14.6\%;$$

The coefficient of variation:

$$v = \frac{\sigma}{\overline{M}/\overline{M}_{calc}} = 0.1382.$$

Such indicators of statistical processing showed high reliability of developed technique for determination of residual bearing capacity of the damaged T-shaped beams.

Table 4.2

The calculation results by the proposed method

Label of the beam	M, kNm	M_{calc}, kNm	$\frac{M_{calc}}{M}$
B 1	20	18.4	0.92
B 2	10	12.92	1.29
B 3	32.5	37.36	1.14
B 4	26.25	24.5	0.92
B 5	23.75	20.81	0.88
B 6	32.5	24.3	0.75
B 7	27.5	32.12	1.17
B 8	18.75	15.5	0.83
B 9	23.75	22.12	0.93
B 10	24.5	25.9	1.05
B 11	22.5	20.34	0.90
B 12	29.5	28.3	0.96
B 13	22.5	19.8	0.88
B 14	26.25	24.3	0.93
B 15	27.5	23.6	0.86

Summing up it is possible to make a sequential algorithm for any of the design cases, following which it is possible to find the unknowns of the task (Fig. 4.16).

4.3. Conclusions for the Chapter

1. The defined in the software system LIRA®-SAPR 2013 values of residual bearing capacity of bent T-shaped elements, damaged in the process of operation, correlate well with experimental data. For determining the destructive moment of the action of external forces and modeling NDS one of the latest versions of software was used, which allows to perform calculations of building constructions in nonlinear cases, corresponding to the current regulations.

2. Simulation shows good convergence of the results with laboratory tests of the error is about 6 %, the percentage of variation was 5.4 %. However, one of the main disadvantages of this calculation is a time consuming process from the point of view of designing and further iterative calculation to determine and clarify the bearing capacity of samples.

3. The current state building codes do not give recommendations for the calculation of damaged T-shaped beams, therefore, there was developed a calculation method which allows determining the residual bearing capacity of such samples. The basic backgrounds of the calculation were created; the design cases and the differences between them were described. A calculation algorithm was created by which it is possible to find all unknowns of the task.

4. The developed technique showed good convergence with results of laboratory tests – the margin of error was ≈14%, the percentage of variation 13.82 %. Such indicators show the possibility of using the technique in the reconstruction of individual structural elements of buildings and structures.

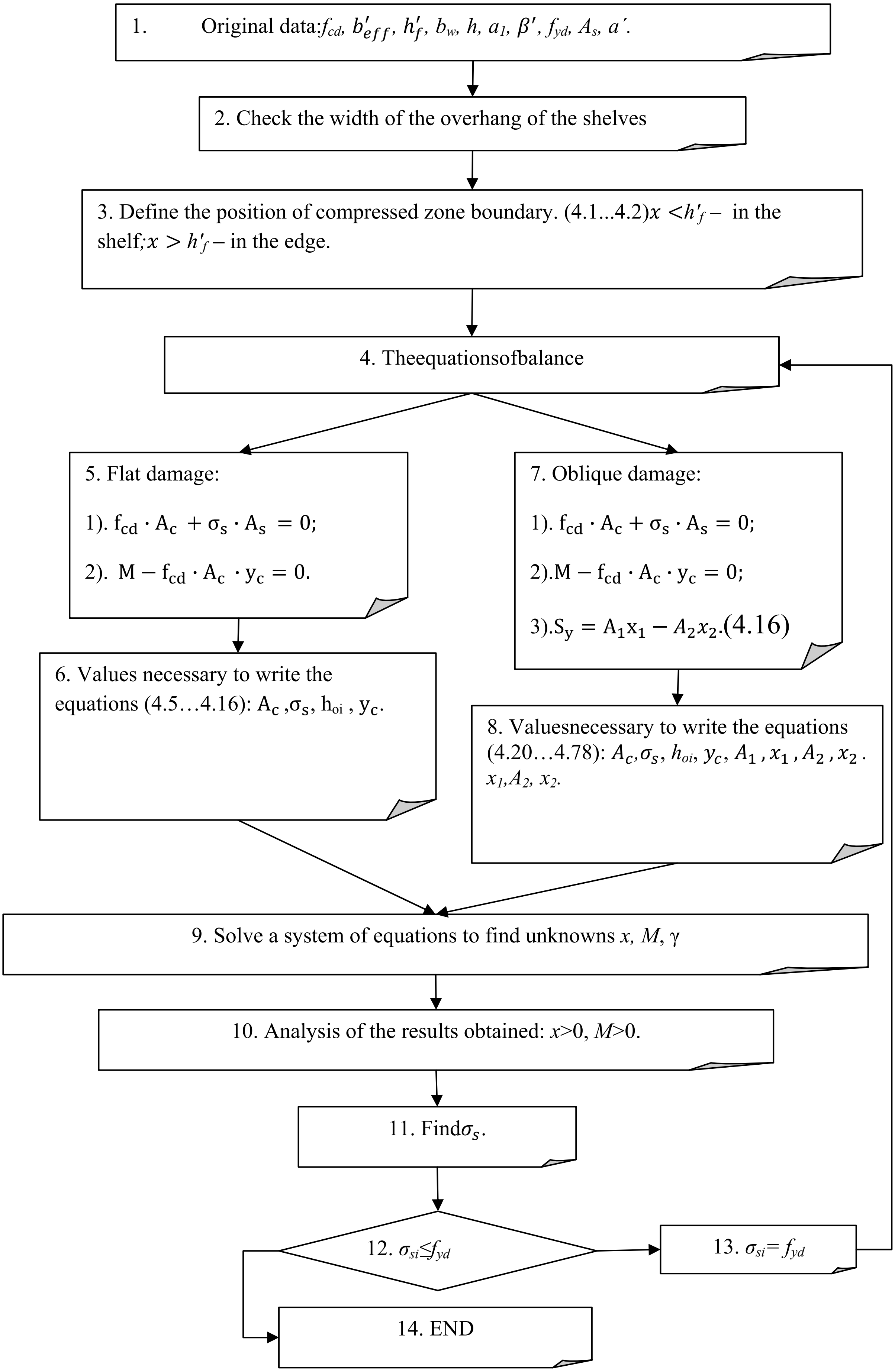


Fig. 4.16.The algorithm for finding the unknown quantities.

CHAPTER 5.

PRACTICAL APPLICATION OF THE METHOD OF CALCULATION OF DAMAGED T-SHAPED BEAMS

5.1. Examples of calculation of damaged experimental beams

Consider a calculation of an element with flat-front damage by the example of a sample B 13.

A length of T-shaped beam is 2.5 m, its height is $h = 0.25$ m. Thickness of the remaining shelf ish'_f= 0.03 m. The depth of a damage of the shelf is a_1=0.03 m (Fig. 5.1, a). Shelf width isb'_{eff}= 0.4 m.The strength of concrete is$f_{cd} = 19\ MPa$.Thereinforcement comprises one reinforcing bar A400C Ø16 mm (f_{yd}= 560 MPa). Protective layer is $a = 20$ mm. It is necessary to determine the bearing capacity of a damaged T-shaped beam.

The method of calculation of T-shaped beams with a flat damage, which was stated in the previous chapter and the calculationalgorithm, can be used to solve the task.

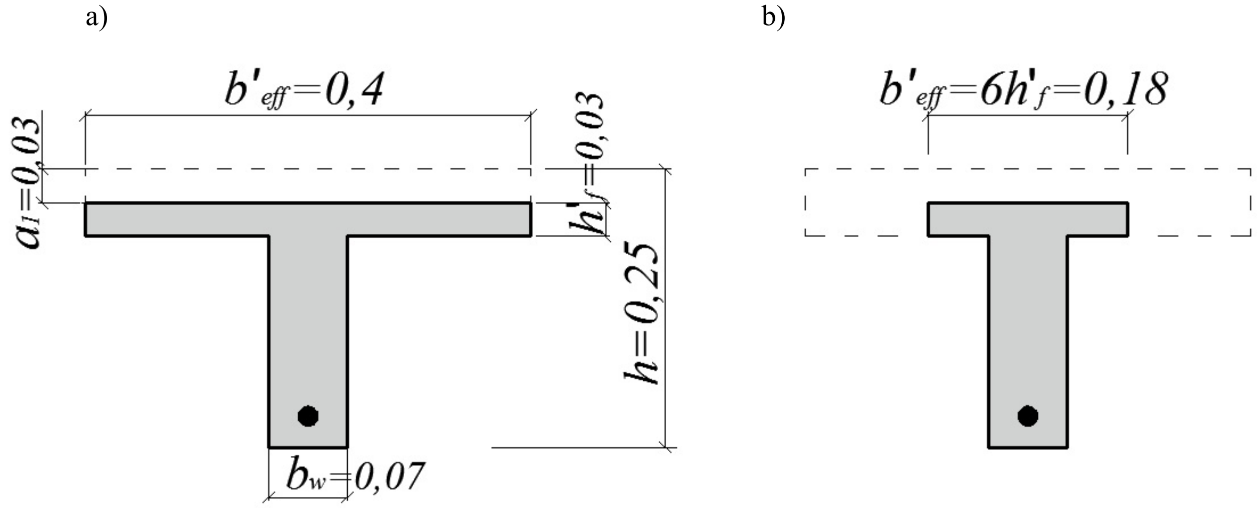


Fig. 5.1.The calculation of the damaged T-shaped beam B 13: a – initial data; b – a design scheme taking into account a check of the width of overhang of the shelf.

The amount of overhang of the shelf should be checked first:

$$h'_f \geq 0.1h; \quad (5.1)$$

$$0.03 > 0.1 \times 0{,}25 = 0.025. \quad (5.2)$$

Assume that $b'_{eff} = 6h'_f = 6 \times 0.03 = 0.18$ m (Fig. 5.1, b).

Thus, overhangs are taken into account not fully.

The position of the boundary of the compressed zone can be determined (4.1…4.2).

$$A_s = \pi \frac{d^2}{4} = 3.14 \times \frac{0.016^2}{4} = 0.000201 \text{ m}^2; \quad (5.3)$$

$$x = \frac{f_{yd} \times A_s}{f_{cd} \times b'_{eff}} = \frac{560 \times 0.000201}{19 \times 0.18} = 0.033 \text{ m}. \quad (5.4)$$

That is the boundary of the compressed zone lies within the edge.

To find unknown quantities the system of equations is required. As there are two unknown quantities (a moment and a height of the compressed zone), two equations are needed to find them (4.3…4.4), namely:

$$\begin{cases} f_{cd} \times A_c + \sigma_s \times A_s = 0; \\ M - f_{cd} \times A_c \times y_c = 0. \end{cases} \quad (5.5)$$

To write down the system of equations the required quantities should be expressed. As the boundary of the compressed zone lies within the edge, the area of the compressed zone will consist of areas of two rectangles (Fig. 5.2, b):

$$A_c = b'_{eff} \times h'_f + b_w \times (x - h'_f); \quad (5.5)$$

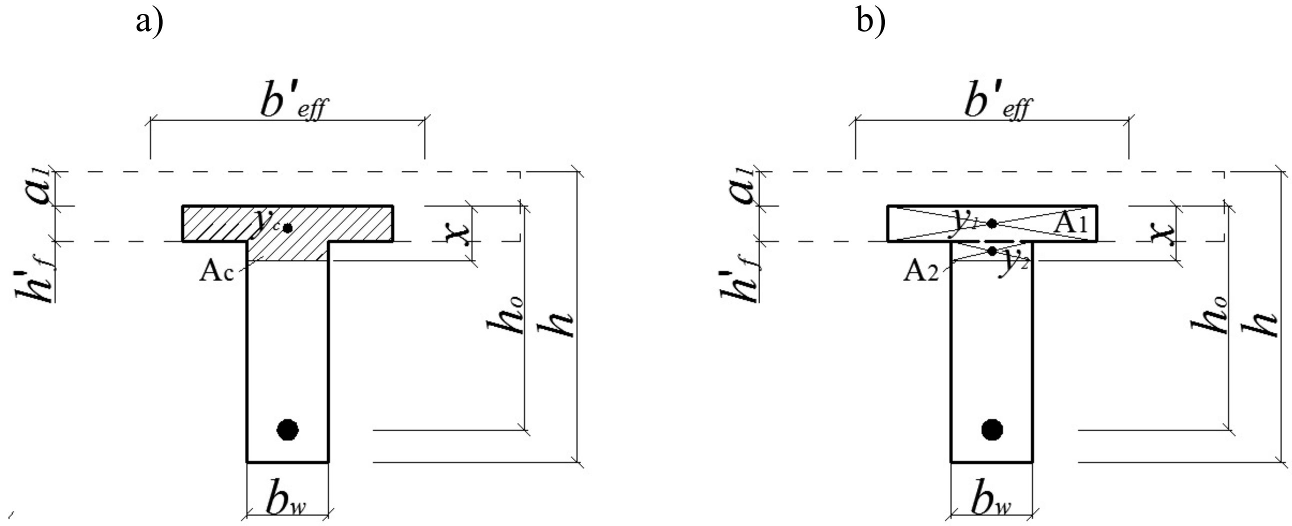


Fig. 5.2. Determination of the area of the compressed zone: a – the area of the compressed zone A_c and the center of mass y_c; b – a division of the compressed zone into simple figures.

Stresses σ_s in a stretched reinforcement should be defined according to the known empirical dependence (4.7):

$$\sigma_{si} = \frac{\sigma_{sc,u}}{1-\frac{\omega}{1.1}}\left(\frac{\omega}{\xi_i} - 1\right), \quad (5.6)$$

where:

$$\omega = \alpha - 0.008 \times f_{cd} = 0{,}85 - 0.008 \times 19 = 0.698; \quad (5.7)$$

$$\sigma_{sc,u} = 400\ MPa.$$

A quantity h_0:

$$h_0 = h - \left(a + \frac{d}{2}\right) - a' = 0.25 - \left(0{,}02 + \frac{0.016}{2}\right) - 0{,}03 = 0.192; \quad (5.8)$$

$$\xi_i = \frac{\mathrm{x}}{\mathrm{h_o}} = \frac{\mathrm{x}}{0.192}; \quad (5.9)$$

A coordinate of the center of mass of the compressed concrete y_c(Fig. 5.2), which is a part of the second equation of the system, can be expressed in the following way:

$$y_c = \frac{A_1 \cdot y_1 + A_2 \cdot y_2}{A_1 + A_2}, \quad (5.10)$$

where:

$$A_1 = b'_{eff} \times h'_f; \quad (5.11)$$

$$A_2 = b_w \times (x - h'_f); \quad (5.12)$$

$$y_1 = h_0 - \frac{h'_f}{2}; \quad (5.13)$$

$$y_2 = h_0 - x + \frac{(x - h'_f)}{2}. \quad (5.14)$$

After a substitution of all the quantities in the system of equations, the following expressions were received:

1). $f_{cd} \times (b'_{eff} \times h'_f + b_w \times \left(x - h'_f\right)) + \frac{\sigma_{sc,u}}{1-\frac{\omega}{1.1}}\left(\frac{\omega \cdot h_o}{x} - 1\right) \times A_s = 0;$ (5.15)

2). $M - f_{cd} \times (b'_{eff} \times h'_f + b_w \times \left(x - h'_f\right)) \times$ (5.16)

$$\times \frac{\left(b'_{eff} \times h'_f\right) \times \left(h_0 - \frac{h'_f}{2}\right) + \left(b_w \times \left(x - h'_f\right)\right) \times \left(h_0 - x + \frac{(x - h'_f)}{2}\right)}{b'_{eff} \times h'_f + b_w \times \left(x - h'_f\right)} = 0.$$

In numerical form:

1). $19 \times (0.18 \times 0.03) + 0.07 \times (x - 0.03)) + \frac{400}{1-\frac{0.698}{1.1}}\left(\frac{0.698 \times 0.192}{x} - 1\right) \times 0.000201 \quad = 0;$

2). $M - 19 \times (0.18 \times 0.03 + 0.07 \times (x - 0.03)) \times$

$$\times \frac{(0.18 \times 0.03) \times \left(0.192 - \frac{0.03}{2}\right) + \left(0.07 \times (x - 0.03)\right) \times \left(0.192 - x + \frac{(x-0.03)}{2}\right)}{0.18 \times 0.03 + 0.07 \times (x - 0.03)} = 0.$$

The system of equations can be solved in different ways. A Newton's method was used to solve the problems in this thesis.

Thus, after solving the system of equations values of the unknown quantities *x* and *M* were received.It is necessary to take into account that the values of the sought quantities are positive and cannot be less than zero, therefore, if there are several pairs of roots, the appropriate roots should be chosen.

$$\begin{cases} \mathrm{x} = 0.07665\ \mathrm{m}; \\ \mathrm{M} = 0.02676\ \mathrm{MNm} = 26.76\ \mathrm{kNm}. \end{cases}$$

$$\sigma_{\mathrm{si}} = \frac{400}{1 - \frac{0.698}{1.1}}\left(\frac{0.698 \times 0.192}{0.07665} - 1\right) = 819.14\ \mathrm{MPa} > \mathrm{f_{yd}} = 560\ \mathrm{MPa}.$$

Since the tensile stresses in the reinforcing bar are more than the limiting, then accept that they are equal to the limiting and recalculate the system:

1). $19 \times (0.18 \times 0.03) + 0.07 \times (x - 0.03)) + 560 \times 0.000201 \quad = 0;$

2). $M - 19 \times (0.18 \times 0.03 + 0.07 \times (x - 0.03)) \times$

$$\times \frac{(0.18 \times 0.03) \times \left(0.192 - \frac{0.03}{2}\right) + \left(0.07 \times (x - 0.03)\right) \times \left(0.192 - x + \frac{(x-0.03)}{2}\right)}{0.18 \times 0.03 + 0.07 \times (x - 0.03)} = 0;$$

Found roots:

$$\begin{cases} x = 0.037\ \mathrm{m}; \\ M = 0.01974\ \mathrm{MNm} = 19.74\ \mathrm{kNm}. \end{cases}$$

Whereupon the calculation is complete.

Consider a calculation of an element with an oblique front damage by the example of a sample B 10.

A length of the T-shaped beam is 2.5 m, its height is h = 0.25 m. Shelf thickness ish'_f= 0.06 m. Parameters of a damage of the shelf: the depth of a damage isa_1 = 0.03 m, the angle of a damage is β = 45˚. The width of the remaining shelf isb'_{eff}= 0.235 m (Fig. 5.3, a).The strength of concrete is$f_{cd} = 27.7\ MPa$.The reinforcement comprises a reinforcing bar which is A400C Ø16 mm in a rib (f_{yd} = 560 MPa). Protective layer is a = 20 mm.

It is necessary to determine the residual bearing capacity of a damaged T-shaped beam.

The method of calculation of T-shaped beams with an oblique damage, which was stated in the fourth chapter and the calculational gorithm, can be used to solve the task.

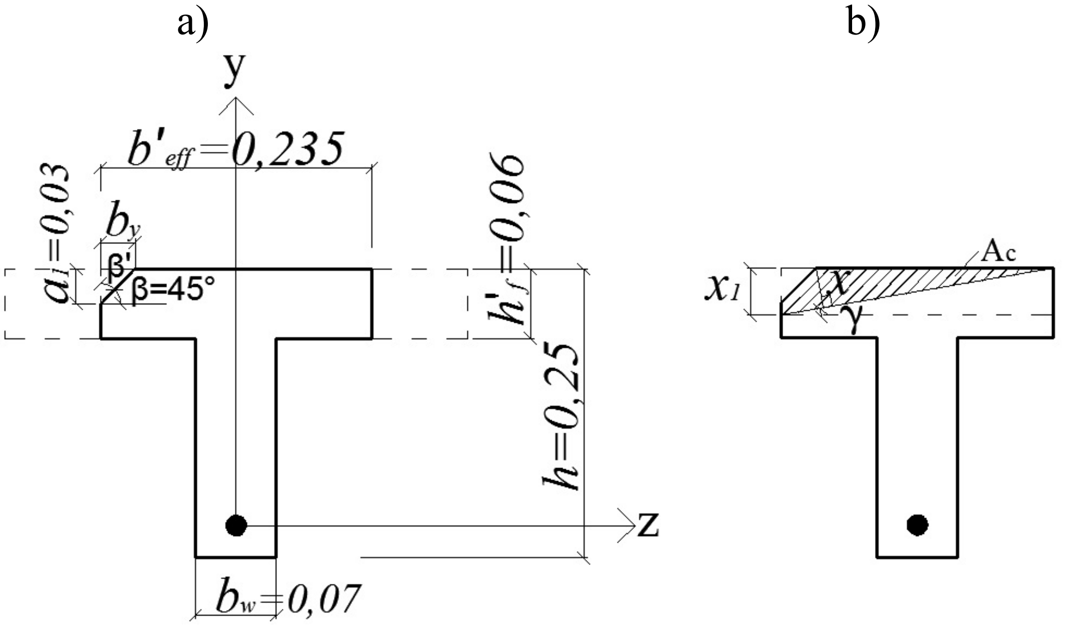


Fig. 5.3.The calculation of the damaged T-shaped beam B 10: a – initial data; b – a design scheme with a predetermined outline of the compressed zone.

The width of overhang of the shelf should be checked:

$$h_f' \geq 0.1h;$$

$$0.06 > 0.1 \cdot 0.25 = 0.025.$$

Under this condition the width of the shelf should not exceed $b_{eff}' = 6h_f'$. The condition is satisfied; the overhang of the shelf is considered fully.

The definition of the position of the boundary of the compressed zone is required. Let the neutral line pass in the shelf, as shown on Fig. 5.3, b.

To find unknown quantities the system of equations is required. As there are three unknown quantities (a moment$M$, a height of the compressed zone x, a tilt angle of the neutral line γ), three equations are needed to find them (4.17….4.19):

$$\begin{cases} f_{cd} \times A_c + \sigma_s \times A_s = 0; \\ M - f_{cd} \times A_c \times y_c = 0; \\ S_y = A_3 x_3 - A_4 x_4. \end{cases} \qquad (5.17)$$

To write down the system of equations the required quantities should be expressed.

Since the angle of a damage is β = 45˚ (Fig. 5.3, a), then β' = 45˚ too, in other words, a triangle in the damage zone is equilateral, and thus, $b_y = a_1$=0.03 m. The area of the compressed zone equals to the difference between the areas of two triangles (Fig. 5.4):

$$A_c = \frac{1}{2} x_1 \times b'_{eff} - \frac{1}{2} a_1 \times b_y.$$

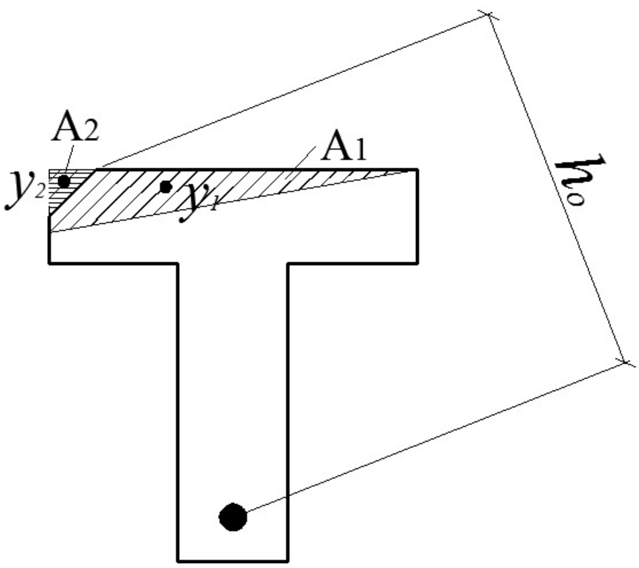


Fig. 5.4.A division of the compressed zone into simple figures.

Stresses σ_s in a stretched reinforcement should be defined according to the formula (4.7):

$$\sigma_{si} = \frac{\sigma_{sc,u}}{1 - \frac{\omega}{1.1}} \left(\frac{\omega}{\xi_i} - 1 \right),$$

where:

$$\omega = \alpha - 0.008$$

$$f_{cd} = 0.85 - 0.008 \times 27.7 = 0.6284;$$

$$\sigma_{sc,u} = 400\ MPa.$$

A quantity h_0(4.28):

$$h_0 = \left(h - \left(a + \frac{d}{2}\right) + \left(\frac{b'_{eff}}{2} - b_y\right) \times \text{tg}\gamma\right) \times \cos\gamma$$

$$= (0.25 - \left(0.02 + \frac{0.016}{2}\right) +$$

$$+ \left(\frac{0.235}{2} - 0.03\right) \times \text{tg}\gamma) \times \cos\gamma = (0.222 + 0.0875 \times \text{tg}\gamma) \times \cos\gamma;$$

$$\xi_i = \frac{x_1}{h_o} = \frac{x_1}{(0.222 + 0.0875 \cdot tg\gamma) \cdot cos\gamma}.$$

Coordinates of the center of mass of the compressed concrete x_c, y_c can be expressed in the following way:

$$x_c = \frac{A_1 \times x_1 - A_2 \times x_2}{A_1 - A_2}; \tag{5.18}$$

$$y_c = \frac{A_1 \times y_1 - A_2 \times y_2}{A_1 - A_2}; \tag{5.19}$$

where (Fig. 5.4):

$$A_1 = \frac{1}{2} b'_{eff} \times x_1; \tag{5.20}$$

$$A_2 = \frac{1}{2} \times b_y \times a_1; \tag{5.21}$$

$$x_1 = \frac{b'_{eff}}{3}; \tag{5.22}$$

$$x_2 = \frac{b_y}{3}; \tag{5.23}$$

$$y_1 = h - (a + \frac{d}{2}) - \frac{x_1}{3}; \tag{5.24}$$

$$y_2 = h - (a + \frac{d}{2}) - \frac{a_1}{3}. \tag{5.25}$$

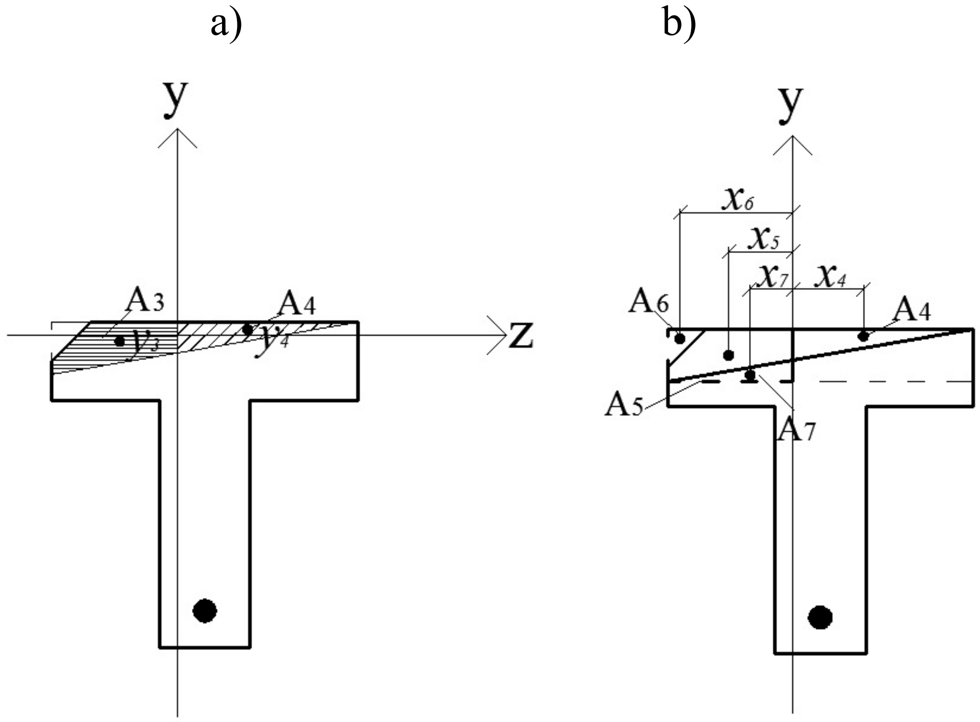


Fig. 5.5. A static moment relatively to the main axis *y*: a – a balance of the parts of a section relatively to the axis *y*; b – simple compound figures.

The third equation (4.19) expresses a static moment of the compressed zone of concreterelatively to the axis *y* which can be defined by dividing into simple figures (Fig. 5.5).

$$S_y = A_3 x_3 - A_4 x_4; \tag{5.26}$$

$$A_3 = A_5 - A_6 - A_7; \tag{5.27}$$

$$x_3 = \frac{A_5 \times x_5 - A_6 \times y_6 - A_7 \times y_7}{A_5 - A_6 - A_7}. \tag{5.28}$$

Where:

$$A_5 = x_c \times a_1; \tag{5.29}$$

$$A_6 = \frac{1}{2} \times b_y \times a_1; \tag{5.30}$$

$$A_7 = \frac{1}{2} \times x_c \times x_c \times tg\gamma; \tag{5.31}$$

$$x_5 = \frac{x_c}{2}; \tag{5.32}$$

$$x_6 = (x_c - \frac{b_y}{3}); \tag{5.33}$$

$$x_7 = \frac{x_c}{3}; \tag{5.34}$$

$$A_4 = \frac{1}{2} \times \left(b'_{eff} - x_c\right) \times (a_1 - x_c \times tg\gamma); \tag{5.35}$$

$$x_4 = \frac{(b'_{eff} - x_c)}{3}. \tag{5.36}$$

After a substitution of all the quantities in the system of equations, the following expressions were received:

1). $f_{cd} \times (\frac{1}{2}x_1 \times b'_{eff} - \frac{1}{2} \times \ b_y \times a_1) + \frac{\sigma_{sc,u}}{1-\frac{\omega}{1.1}}\left(\frac{\omega \times h_o}{x} - 1\right) \times A_s = 0;$ (5.27)

2). $M - f_{cd} \times (\frac{1}{2}x_1 \times b'_{eff} - \frac{1}{2} \times \ b_y \times a_1 \times$ (5.28)

$$\times \frac{\left(\frac{1}{2}x_1 \times b'_{eff}\right) \times \left(h - (a + \frac{d}{2}) - \frac{x_1}{3}\right) - \left(\frac{1}{2} \times \ b_y \times a_1\right) \times \left(h - (a + \frac{d}{2}) - \frac{a_1}{3}\right)}{\frac{1}{2}x_1 \times b'_{eff} - \frac{1}{2} \times \ b_y \times a_1} = 0;$$

3). $\left(x_c \times a_1 - \frac{1}{2} \times b_y \times a_1 - \frac{1}{2} \times x_c \times x_c \times tg\gamma\right) \times$ (5.29)

$$\times \left(\frac{x_c \times a_1 \times \frac{x_c}{2} - \frac{1}{2} \times b_y \times a_1 \times \left(x_c - \frac{b_y}{3}\right) - \frac{1}{2} \times x_c \times x_c \times tg\gamma \times \frac{x_c}{3}}{x_c \times a_1 - \frac{1}{2} \times b_y \times a_1 - \frac{1}{2} \times x_c \times x_c \times tg\gamma}\right) -$$

$$-\frac{1}{2} \times \left(b'_{eff} - x_c\right) \times (a_1 - x_c \times tg\gamma) \times \frac{(b'_{eff} \ - x_c)}{3} = 0.$$

In numerical form:

$$). \ 27.7 \times \left(\frac{1}{2}x_1 \times 0.235 - \frac{1}{2} \times 0.03^2\right) +$$

$$+\frac{400}{1 - \frac{0.6284}{1.1}}\left(\frac{0.6284 \times \left((0.222 + 0.0875 \times tg\gamma) \times cos\gamma\right)}{x} - 1\right) \times 0.000201 = 0;$$

2). $M - 27.7 \times \left(\frac{1}{2}x_1 \times 0.235 - \frac{1}{2} \times 0.03^2\right) \times$

$$\times \frac{\left(\frac{1}{2}x_1 \times 0.235\right) \times \left((0.25 - \left(0.2 + \frac{0.016}{2}\right)) - \frac{x_1}{3}\right) - \left(\frac{1}{2} \times 0.03^2\right) \times \left((0.25 - \left(0.02 + \frac{0.016}{2}\right)) - \frac{0.03}{3}\right)}{\frac{1}{2}x_1 \times 0.235 \quad - \frac{1}{2} \times 0.03^2}$$

$= 0;$

3).

$$\left(\left(\frac{\frac{1}{2}0.235\times x_1\times\frac{0.235}{3}-\frac{1}{2}\times 0.03\times 0.03\times\frac{0.03}{3}}{\frac{1}{2}0.235\times x_1-\frac{1}{2}\times 0.03\times 0.03}\right)\times 0.03-\frac{1}{2}\times 0.03\times 0.03-\frac{1}{2}\times\right.$$

$$\left.\left(\frac{\frac{1}{2}0.235\times x_1\times\frac{0.235}{3}-\frac{1}{2}\times 0.03\times 0.03\times\frac{0.03}{3}}{\frac{1}{2}0.235\times x_1-\frac{1}{2}\times 0.03\times 0.03}\right)^2\times tg\gamma\right)\times\left(\left(\frac{\frac{1}{2}0.235\times x_1\times\frac{0.235}{3}-\frac{1}{2}\times 0.03\times 0.03\times\frac{0.03}{3}}{\frac{1}{2}0.235\times x_1-\frac{1}{2}\times 0.03\times 0.03}\right)\times 0.03\times\ 0.5\times\right.$$

$$\left(\frac{\frac{1}{2}0.235\times x_1\times\frac{0.235}{3}-\frac{1}{2}\times 0.03\times 0.03\times\frac{0.03}{3}}{\frac{1}{2}0.235\times x_1-\frac{1}{2}\times 0.03\times 0.03}\right)-$$

$$-\frac{1}{2}\times 0.03\times 0.03\times\left(\left(\frac{\frac{1}{2}0.235\times x_1\times\frac{0.235}{3}-\frac{1}{2}\times 0.03\times 0.03\times\frac{0.03}{3}}{\frac{1}{2}\times 0.235\times x_1-\frac{1}{2}\times 0.03\times 0.03}\right)-\frac{0.03}{3}\right)-\frac{1}{2}\times$$

$$\times\left(\frac{\frac{1}{2}0.235\times x_1\times\frac{0.235}{3}-\frac{1}{2}\times 0.03\times 0.03\times\frac{0.03}{3}}{\frac{1}{2}\times 0.235\times x_1-\frac{1}{2}\times 0.03\times 0.03}\right)^2\times tg\gamma\times$$

$$\times\frac{\left(\frac{\frac{1}{2}\times 0.235\times x_1\times\frac{0.235}{3}-\frac{1}{2}\times 0.03\times 0.03\times\frac{0.03}{3}}{\frac{1}{2}\times 0.235\times x_1-\frac{1}{2}\times 0.03\times 0.03}\right)}{3})//(\left(\frac{\frac{1}{2}\times 0.235\times x_1\times\frac{0.235}{3}-\frac{1}{2}\times 0.03\times 0.03\times\frac{0.03}{3}}{\frac{1}{2}\times 0.235\times x_1-\frac{1}{2}\times 0.03\times 0.03}\right)\times 0.03-$$

$$-\frac{1}{2}\times 0.03\times 0.03-\frac{1}{2}\times\left(\frac{\frac{1}{2}\times 0.235\times x_1\times\frac{0.235}{3}-\frac{1}{2}\times 0.03\times 0.03\times\frac{0.03}{3}}{\frac{1}{2}\times 0.235\times x_1-\frac{1}{2}\times 0.03\times 0.03}\right)^2\times tg\gamma)-$$

$$-\frac{1}{2}\times\left(0.235-\left(\frac{\frac{1}{2}\times 0.235\times x_1\times\frac{0.235}{3}-\frac{1}{2}\times 0.03\times 0.03\times\frac{0.03}{3}}{\frac{1}{2}\times 0.235\times x_1-\frac{1}{2}\times 0.03\times 0.03}\right)\right)\times(0.03-$$

$$-\left(\frac{\frac{1}{2}\times 0.235\times x_1\times\frac{0.235}{3}-\frac{1}{2}\times 0.03\times 0.03\times\frac{0.03}{3}}{\frac{1}{2}\times 0.235\times x_1-\frac{1}{2}\times 0.03\times 0.03}\right)\times tg\gamma)\times\times\frac{(0.235-\left(\frac{\frac{1}{2}\times 0.235\times x_1\times\frac{0.235}{3}-\frac{1}{2}\times 0.03\times 0.03\times\frac{0.03}{3}}{\frac{1}{2}\times 0.235\times x_1-\frac{1}{2}\times 0.03\times 0.03}\right))}{3}=0.$$

This system of equations does not have any roots that satisfy the requirements of the task ($x>0$and $M>0$) when it is solved in numerical form. It means that the accepted compressed zone is not true.

Therefore, the recalculation of the problem should be done with another outline of the compressed zone (Fig. 5.6, a).

The system of equations remains the same:

$$\begin{cases} f_{cd} \times A_c + \sigma_s \times A_s = 0; \\ M - f_{cd} \times A_c \times y_c = 0; \\ S_y = A_4 x_4 - A_5 x_5. \end{cases} \quad (5.30)$$

The area of the compressed zone is equal to the difference between the simple figures (Fig. 5.6, b):

$$A_c = a_1 \times b'_{eff} - \frac{1}{2} a_1 \times b_y - \frac{1}{2} \times {b'_{eff}}^2 \times tg\gamma. \quad (5.31)$$

Coordinates of the center of mass of the compressed concrete x_c, y_c:

$$x_c = \frac{A_1 \times x_1 - A_2 \times x_2 - A_3 \times x_3}{A_1 - A_2 - A_3}; \quad (5.32)$$

$$y_c = \frac{A_1 \times y_1 - A_2 \times y_2 - A_3 \times y_3}{A_1 - A_2 - A_3}, \quad (5.33)$$

where (Fig. 5.6, b):

$$A_1 = a_1 \times b'_{eff}; (5.34)$$

$$A_2 = \frac{1}{2} a_1 \times b_y; \quad (5.35)$$

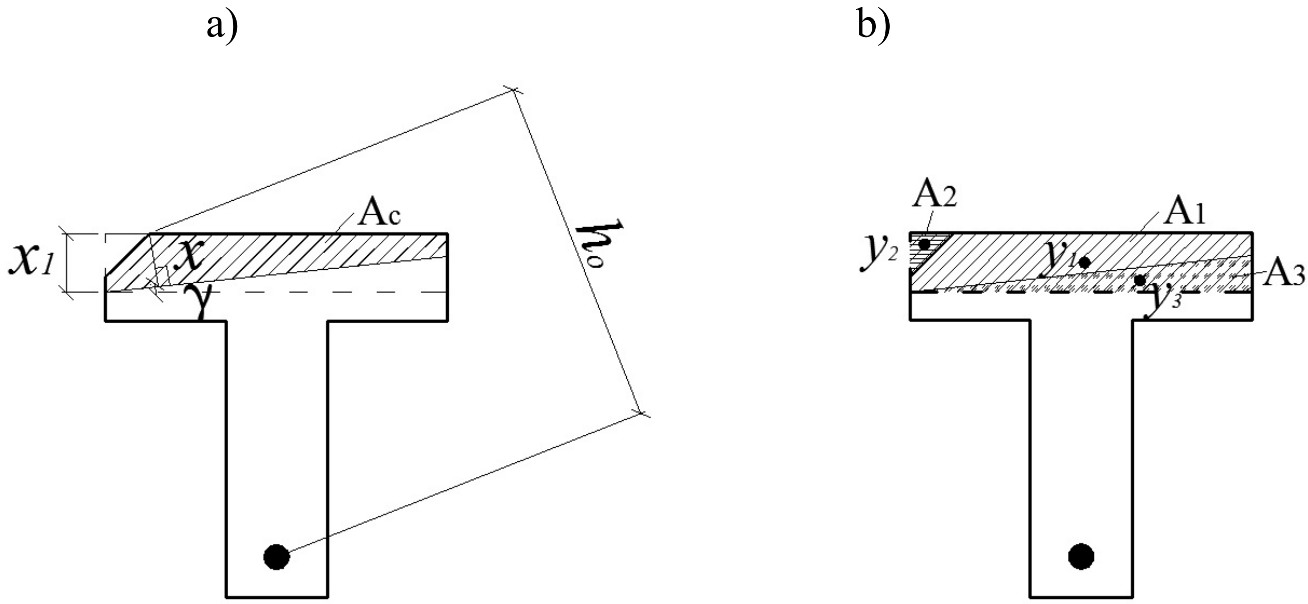


Fig. 5.6. Determination of the area of the compression zone: a – an accepted outline of the compressed zone; b – a division of the compressed zone into simple figures.

$$A_3 = \frac{1}{2} {b'_{eff}}^2 \times tg\gamma; \quad (5.36)$$

$$x_1 = \frac{b'_{eff}}{2}; \quad (5.37)$$

$$x_2 = \frac{b_y}{3}; \quad (5.38)$$

$$x_3 = \frac{3b'_{eff}}{2}; \quad (5.39)$$

$$y_1 = h - (a + \frac{d}{2}) - \frac{a_1}{2}; \quad (5.40)$$

$$y_2 = h - \left(a + \frac{d}{2}\right) - \frac{a_1}{3}; \quad (5.40)$$

$$y_3 = h - \left(a + \frac{d}{2}\right) - a_1 - \frac{b'_{eff} \times tg\gamma}{3}. \quad (5.41)$$

A static moment of the compressed zone of the concrete relatively to the axis *y* can be defined by dividing into simple figures (Fig. 5.7).

$$A_4 = A_6 - A_7 - A_8; \quad (5.42)$$

$$x_4 = \frac{A_6 \times x_6 - A_7 \times y_7 - A_8 \times y_8}{A_6 - A_7 - A_8}; \quad (5.43)$$

$$A_4 = A_9 - A_{10}; \quad (5.44)$$

$$x_4 = \frac{A_9 \times x_9 - A_{10} \times y_{10}}{A_9 - A_{10}}. \quad (5.45)$$

Where:

$$A_6 = x_c \times a_1; \quad (5.46)$$

$$A_7 = \frac{1}{2} b_y \times a_1; \quad (5.47)$$

$$A_8 = \frac{1}{2} x_c \times x_c \times tg\gamma; \quad (5.48)$$

$$A_9 = \frac{1}{2} \times \left(b'_{eff} - x_c\right) \times (a_1 - x_c \times tg\gamma); \quad (5.49)$$

$$A_{10} = \frac{1}{2} \times \left(b'_{eff} - x_c\right) \times \left(b'_{eff} - x_c\right) \times tg\gamma; \quad (5.50)$$

a) b)

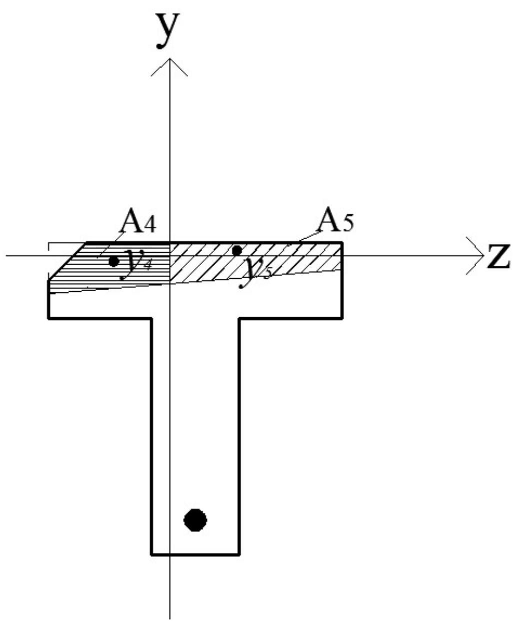


Fig. 5.7. A static moment relatively to the main axis y: a – a balance of the parts of a section relatively to the axis y; b – simple compound figures.

$$x_6 = \frac{x_c}{2}; \tag{5.51}$$

$$x_7 = (x_c - \frac{b_y}{3}); \tag{5.52}$$

$$x_8 = \frac{x_c}{3}; \tag{5.53}$$

$$x_9 = \frac{(b'_{eff} - x_c)}{2}; \tag{5.54}$$

$$x_{10} = \frac{2 \times (b'_{eff} - x_c)}{3}. \tag{5.55}$$

After a substitution of all the quantities in the system of equations, the following expressions were received:

1). $f_{cd} \times (a_1 \times b'_{eff} - \frac{1}{2} a_1 \times b_y - \frac{1}{2} \times {b'_{eff}}^2 \times tg\gamma) + \frac{\sigma_{sc,u}}{1 - \frac{\omega}{1,1}} \left(\frac{\omega \times h_o}{x} - 1 \right) \times A_s = 0;$

2).$M - f_{cd} \times (a_1 \times b'_{eff} - \frac{1}{2} a_1 \times b_y - \frac{1}{2} {b'_{eff}}^2 \times tg\gamma) \times$

$$\times ((a_1 \times b'_{eff}) \times \left(h - \left(a + \frac{d}{2}\right) - \frac{a_1}{2}\right) - \left(\frac{1}{2} a_1 \times b_y\right) \times \left(h - \left(a + \frac{d}{2}\right) - \frac{a_1}{3}\right) -$$

$$-\left(\frac{1}{2}\times {b'_{eff}}^{2}\times tg\gamma\right)\times$$

$$\times\left(h-\left(a+\frac{d}{2}\right)-a_1-\frac{b'_{eff}\ \times tg\gamma}{3}\right))/(a_1\times b'_{eff}-\frac{1}{2}a_1\times b_y-\frac{1}{2}\times {b'_{eff}}^{2}\times tg\gamma)$$
$$=0;$$

3). $$\left(x_c\times a_1-\frac{1}{2}b_y\times a_1-\frac{1}{2}x_c\times x_c\times tg\gamma\right)\times$$
$$\left(\frac{x_c\times a_1\times\frac{x_c}{2}-\frac{1}{2}b_y\times a_1\times\left(x_c-\frac{b_y}{3}\right)-\frac{1}{2}x_c\times x_c\times tg\gamma\times\frac{x_c}{3}}{x_c\times a_1-\frac{1}{2}\times b_y\times a_1-\frac{1}{2}\times x_c\times x_c\times tg\gamma}\right)-$$

$$-\left(\frac{1}{2}\left(b'_{eff}-x_c\right)\times(a_1-x_c\times tg\gamma)-\frac{1}{2}\left(b'_{eff}-x_c\right)\times\left(b'_{eff}-x_c\right)\times tg\gamma\right)\times$$

$$\times\left(\frac{\frac{1}{2}\left(b'_{eff}-x_c\right)\times(a_1-x_c\times tg\gamma)\frac{(b'_{eff}-x_c)}{2}-\frac{1}{2}\left(b'_{eff}-x_c\right)\times\left(b'_{eff}-x_c\right)\times tg\gamma\frac{2(b'_{eff}-x_c)}{3}}{\frac{1}{2}\left(b'_{eff}-x_c\right)\times(a_1-x_c\times tg\gamma)-\frac{1}{2}\left(b'_{eff}-x_c\right)\times\left(b'_{eff}-x_c\right)\times tg\gamma}\right)=0.$$

Numerically equations are expressed as following:

1). $$27.7\times(0.03\times0.235\quad-\frac{1}{2}0.03\times0.03-\frac{1}{2}\times0.235^2\times tg\gamma)+$$

$$+\frac{400}{1-\frac{0.6284}{1.1}}\left(\frac{0.6284\big((0.222+0.0875\times tg\gamma)\times cos\gamma\big)}{x}-1\right)\times0.000201\quad=0;$$

2). $$M-27.7(0.03\times0.235\quad-\frac{1}{2}0.03\times0.03-\frac{1}{2}0.235^2\times tg\gamma)\times$$

$$\times((0.03\times0.235)\left(0.25-\left(0.02+\frac{0.016}{2}\right)-\frac{0.03}{2}\right)-\left(\frac{1}{2}0.03\times0.03\right)\left(0.25-\left(0.02+\frac{0.016}{2}\right)-\frac{0.03}{3}\right)-$$

$$-\left(\frac{1}{2}\times0.235^2\times tg\gamma\right)\left(0.25-\left(0.02+\frac{0.03}{2}\right)-0.03-\frac{0.235\times tg\gamma}{3}\right))/(0.03\times0.235-$$

$$-\frac{1}{2}0.03\times0.03-\frac{1}{2}0.235^2\times tg\gamma)=0;$$

3). $$(\left(\frac{0.03\times0.235\quad\times\frac{0.235}{2}-\frac{1}{2}\times0.03^2\times\frac{0.03}{3}-\frac{1}{2}\times0.235^2\times tg\gamma\times\frac{3\times0.235}{2}}{0.03\times0.235\quad-\frac{1}{2}\times0.03^2-\frac{1}{2}\times0.235^2\times tg\gamma}\right)\times0.03-$$

$$-\frac{1}{2}\times0.03^2-\frac{1}{2}\left(\frac{0.03\times0.235\quad\frac{0.235}{2}-\frac{1}{2}\times0.03^2\times\frac{0.03}{3}-\frac{1}{2}\times0.235^2\times tg\gamma\times\frac{3\times0.235}{2}}{0.03\times0.235\quad-\frac{1}{2}\times0.03^2-\frac{1}{2}\times0.235^2\times tg\gamma}\right)^2\times tg\gamma)\times$$

$$\times \left(\left(\frac{0.03 \times 0.235 \quad \times \frac{0.235}{2} - \frac{1}{2} \times 0.03^2 \times \frac{0.03}{3} - \frac{1}{2} \times 0.235^2 \times tg\gamma \times \frac{3 \times 0.235}{2}}{0.03 \times 0.235 \quad - \frac{1}{2} \times 0.03^2 - \frac{1}{2} \times 0.235^2 \times tg\gamma}\right) \times 0.03 \times \ 0.5 \times\right.$$

$$\times \left(\frac{0.03 \times 0.235 \quad \times \frac{0.235}{2} - \frac{1}{2} \times 0.03^2 \times \frac{0.03}{3} - \frac{1}{2} \times 0.235^2 \times tg\gamma \times \frac{3 \times 0.235}{2}}{0.03 \times 0.235 \quad - \frac{1}{2} \times 0.03^2 - \frac{1}{2} \times 0.235^2 \times tg\gamma}\right) - \frac{1}{2} \times 0.03^2 \times$$

$$\times \left(\left(\frac{0.03 \times 0.235 \quad \times \frac{0.235}{2} - \frac{1}{2} \times 0.03^2 \times \frac{0.03}{3} - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma \times \frac{3 \times 0.235}{2}}{0.03 \times 0.235 \quad - \frac{1}{2} \times 0.03^2 - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma}\right) - \frac{0.03}{3}\right) -$$

$$- \frac{1}{2} \times \left(\frac{0.03 \times 0.235 \quad \times \frac{0.235}{2} - \frac{1}{2} \times 0.03^2 \times \frac{0.03}{3} - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma \times \frac{3 \times 0.235}{2}}{0.03 \times 0.235 \quad - \frac{1}{2} \times 0.03^2 - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma}\right)^2 \times \mathrm{tg}\gamma \times$$

$$\times \frac{\left(\frac{0.03 \times 0.235 \quad \times \frac{0.235}{2} - \frac{1}{2} \times 0.03^2 \times \frac{0.03}{3} - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma \times \frac{3 \times 0.235}{2}}{0.03 \times 0.235 \quad - \frac{1}{2} \times 0.03^2 - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma}\right)}{3})/$$

$$/\left(\left(\frac{0.03 \times 0.235 \quad \times \frac{0.235}{2} - \frac{1}{2} \times 0.03^2 \times \frac{0.03}{3} - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma \times \frac{3 \times 0.235}{2}}{0.03 \times 0.235 \quad - \frac{1}{2} \times 0.03^2 - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma}\right) \times 0.03 - \frac{1}{2} 0.03^2 -\right.$$

$$- \frac{1}{2} \times \left(\frac{0.03 \times 0.235 \quad \times \frac{0.235}{2} - \frac{1}{2} \times 0.03^2 \times \frac{0.03}{3} - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma \times \frac{3 \times 0.235}{2}}{0.03 \times 0.235 \quad - \frac{1}{2} \times 0.03^2 - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma}\right)^2 \times \mathrm{tg}\gamma) -$$

$$-(\frac{1}{2} \times \left(0.235 - \left(\frac{0.03 \times 0.235 \quad \times \frac{0.235}{2} - \frac{1}{2} \times 0.03^2 \times \frac{0.03}{3} - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma \times \frac{3 \times 0.235}{2}}{0.03 \times 0.235 \quad - \frac{1}{2} \times 0.03^2 - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma}\right)\right) \times$$

$$\times \left(0.03 - \left(\frac{0.03 \times 0.235 \quad \frac{0.235}{2} - \frac{1}{2} \times 0.03^2 \times \frac{0.03}{3} - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma \times \frac{3 \times 0.235}{2}}{0.03 \times 0.235 \quad - \frac{1}{2} \times 0{,}03^2 - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma}\right) \times \mathrm{tg}\gamma\right) -$$

$$- \frac{1}{2} \times \left(0.235 - \left(\frac{0.03 \times 0.235 \quad \times \frac{0.235}{2} - \frac{1}{2} \times 0.03^2 \times \frac{0.03}{3} - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma \times \frac{3 \times 0.235}{2}}{0.03 \times 0.235 \quad - \frac{1}{2} \times 0.03^2 - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma}\right)\right) \times$$

$$\times \left(0.235 - \left(\frac{0.03 \times 0.235 \quad \times \frac{0.235}{2} - \frac{1}{2} 0.03^2 \times \frac{0.03}{3} - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma \times \frac{3 \times 0.235}{2}}{0.03 \times 0.235 \quad - \frac{1}{2} \times 0.03^2 - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma}\right)\right) \times \mathrm{tg}\gamma) \times$$

$$\times (\frac{1}{2} \times \left(0.235 - \left(\frac{0.03 \times 0.235 \quad \times \frac{0.235}{2} - \frac{1}{2} 0.03^2 \times \frac{0.03}{3} - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma \times \frac{3 \times 0.235}{2}}{0.03 \times 0.235 \quad - \frac{1}{2} \times 0.03^2 - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma}\right)\right) \times$$

$$\times \left(0.03 - \left(\frac{0.03 \times 0.235 \quad \times \frac{0.235}{2} - \frac{1}{2} \times 0.03^2 \times \frac{0.03}{3} - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma \times \frac{3 \times 0.235}{2}}{0.03 \times 0.235 \quad - \frac{1}{2} \times 0.03^2 - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma}\right) \times \mathrm{tg}\gamma\right) \times$$

$$\times \frac{\left(0.235 - \left(\frac{0.03 \times 0.235 \quad \times \frac{0.235}{2} - \frac{1}{2} \times 0.03^2 \times \frac{0.03}{3} - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma \times \frac{3 \times 0.235}{2}}{0.03 \times 0.235 \quad - \frac{1}{2} \times 0.03^2 - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma}\right)\right)}{2} -$$

$$-\frac{1}{2} \times \left(0.235 - \left(\frac{0.03 \times 0.235 \quad \times \frac{0.235}{2} - \frac{1}{2} \times 0.03^2 \times \frac{0.03}{3} - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma \times \frac{3 \times 0.235}{2}}{0.03 \times 0.235 \quad - \frac{1}{2} \times 0.03^2 - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma}\right)\right) \times$$

$$\times \left(0.235 - \left(\frac{0.03 \times 0.235 \quad \times \frac{0.235}{2} - \frac{1}{2} \times 0.03^2 \times \frac{0.03}{3} - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma \times \frac{3 \times 0.235}{2}}{0.03 \times 0.235 \quad - \frac{1}{2} \times 0.03^2 - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma}\right)\right) \times$$

$$\times \mathrm{tg}\gamma \times \frac{2 \times \left(0.235 - \left(\frac{0.03 \times 0.235 \quad \times \frac{0.235}{2} - \frac{1}{2} \times 0.03^2 \times \frac{0.03}{3} - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma \times \frac{3 \times 0.235}{2}}{0.03 \times 0.235 \quad - \frac{1}{2} \times 0.03^2 - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma}\right)\right)}{3})/$$

$$/(\frac{1}{2} \times \left(0.235 - \left(\frac{0.03 \times 0.235 \quad \times \frac{0.235}{2} - \frac{1}{2} \times 0.03^2 \times \frac{0.03}{3} - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma \times \frac{3 \times 0.235}{2}}{0.03 \times 0.235 \quad - \frac{1}{2} \times 0.03^2 - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma}\right)\right) \times$$

$$\times \left(0.03 - \left(\frac{0.03 \times 0.235 \quad \times \frac{0.235}{2} - \frac{1}{2} \times 0.03^2 \times \frac{0.03}{3} - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma \times \frac{3 \times 0.235}{2}}{0.03 \times 0.235 \quad - \frac{1}{2} \times 0.03^2 - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma}\right) \times \mathrm{tg}\gamma\right) - \frac{1}{2}$$

$$\times \left(0.235 - \left(\frac{0.03 \times 0.235 \quad \times \frac{0.235}{2} - \frac{1}{2} \times 0.03^2 \times \frac{0.03}{3} - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma \times \frac{3 \times 0.235}{2}}{0.03 \times 0.235 \quad - \frac{1}{2} \times 0.03^2 - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma}\right)\right)$$

$$\times$$

$$\times \left(0.235 - \left(\frac{0.03 \times 0.235 \quad \times \frac{0.235}{2} - \frac{1}{2} 0.03^2 \times \frac{0.03}{3} - \frac{1}{2} 0.235^2 \times \mathrm{tg}\gamma \times \frac{3 \times 0.235}{2}}{0.03 \times 0.235 \quad - \frac{1}{2} \times 0.03^2 - \frac{1}{2} \times 0.235^2 \times \mathrm{tg}\gamma}\right)\right) \times \mathrm{tg}\gamma) = 0.$$

In numerical form the equations become cumbersome, but they are easily solved with the help of a computer (Excel, MATLAB, etc.).

The solution of this system gives the following roots:

$$\begin{cases} x_1 = 0.058 \text{ m}; \\ \text{M} = 0.048 \text{ MNm}; \\ \gamma = 1.037 \text{ rad}. \end{cases}$$

At this value of x the stretching stresses in a reinforcing bar will be equal to $\sigma_s = 1493.23MPa > f_{yd} = 560\ MPa$ according to the formula (4.7). It means the recalculation of the task should be done taking into account that $\sigma_s = f_{yd} = 560\ MPa$:

$$\begin{cases} f_{cd} \times A_c + f_{yd} \times A_s = 0; \\ M - f_{cd} \times A_c \times y_c = 0; \\ S_y = A_4 x_4 - A_5 x_5. \end{cases}$$

As a result, the following roots were obtained:

$$\begin{cases} x_1 = 0.03 \text{ m}; \\ \text{M} = 0.0259 \text{ MNm} = 25.9 \text{ kNm}; \\ \gamma = 0.091 \text{ rad} = 5.21°. \end{cases}$$

Whereupon the calculation is complete.

5.2. The implementation of research results

5.2.1. The implementation of the method of calculation of the damaged elements in the practice of reconstruction. During the reconstruction of the industrial building of the "Glavstroy" LLC at: Odessa, 19th kilometer of the road Staronikolaevskaya, the need to reinforce one of T-shaped beams in the industrial building arose. The beam was damaged due to the mechanical impact of a mobile lifting crane. A checking calculation was performed as described in the thesis because of constraint and also the limitations of fund for a replacement of the construction element and the desirability of not stopping technological processes occurring in this industrial building. This allowed deciding on the method of reinforcement of the T-shaped beam for its normal operation in the future.

The T-shaped beam has normal (non-stressed) reinforcement. It's length is L = 6 m, the section dimensions are b'_{eff} = 0.4 m, h'_f= 0.1 m, b_w= 0.1 m, h= 0.4 m; a = 0.03 m. A shelf of the T-shaped beam has an oblique damage. Parameters of damage that were determined by direct measurement are: $a_1 = 0.1$ m, the angle of spalling of the concrete is β ≈ 38°(Fig. 5.8). Concrete is heavy class C20/25 (f_{cd} = 14.5 MPa); compressed reinforcement is class A 400 C (f_{yd}= 365 MPa); its cross-sectional area is A_s = 0.000314 m^2 (1 ∅ 20).It is necessary to determine the residual bearing capacity of the beam.

The T-shaped beam was designed in accordance with the requirements [88] for accounting the width of the overhang of the shelf; during calculation it is taken into account fully.

In order to analyze the possible value of the bending moment that arise in the damaged T-shaped beam, the bearing capacity of the T-shaped beam with an undamaged section with the same geometric characteristics should be defined.

The position of the boundary of the compressed zone can be determined:

$$x = \frac{f_{yd} \times A_s}{f_{cd} \times b'_{eff}} = \frac{365 \times 0.000314}{14.5 \times 0.4} = 0.02 \text{ m}.$$

In other words, the boundary of the compressed zone passes in the shelf.

$$M = f_{cd} \times A_c \times y_c = f_{cd} \times b'_{eff} \times x \times ((h - \left(a + \frac{d}{2}\right)) - \frac{x}{2});$$

$M = 14.5 \times 0.4 \times 0.02 \times (\left(0.4 - \left(0.03 + \frac{0.02}{2}\right)\right) - \frac{0.02}{2}\Big) = 0.0401$ MNm = 40.1 kNm.

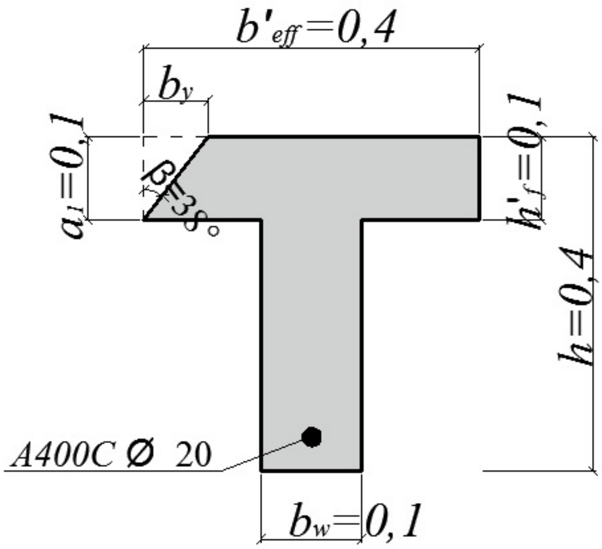


Fig. 5.8.Initial data to calculate the damaged reinforced concrete T-shaped beam.

It is obvious that the damaged beam will stand less than $M = 40{,}1$ kNm.

The calculation of the damaged section should be done next. Assume that the compressed zone passes in the shelf as shown on Fig. 5.9, a.

The system of equations is needed to determine the unknown quantities of the task:

$$\begin{cases} f_{cd} \times A_c + \sigma_s \times A_s = 0; \\ M - f_{cd} \times A_c \times y_c = 0; \\ S_y = A_4 x_4 - A_5 x_5. \end{cases} \tag{5.59}$$

To write down the system of equations the required quantities should be expressed:

$$b_y = a_1 \times tg\beta = 0.1 \times 0.7813 = 0.078; \tag{5.60}$$

$$h_1 = \left(b'_{eff} - b_y + b_1\right) \times tg\gamma. \tag{5.61}$$

Of the similarity of triangles:

$$b_1 = \frac{x_1 \times b_y}{a_1}. \tag{5.62}$$

The area of the compressed zone is equal to the difference between the areas of simple figures (Fig. 5.9, b):

$$A_c = x_1 \times \left(b'_{eff} - b_y + b_1\right) - \frac{1}{2} x_1 \times b_1 \quad - \frac{1}{2}\left(b'_{eff} - b_y + b_1\right) \cdot\times= \tag{5.63}$$

$$= x_1\left(b'_{eff} - b_y + \frac{x_1 \times b_y}{a_1}\right)\frac{1}{2} x_1 \times \frac{x_1 \times b_y}{a_1} \quad \frac{1}{2}\left(b'_{eff} - b_y + \frac{x_1 \times b_y}{a_1}\right)$$

$$\left(b'_{eff} - b_y + + \frac{x_1 \times b_y}{a_1}\right) \times tg\gamma.$$

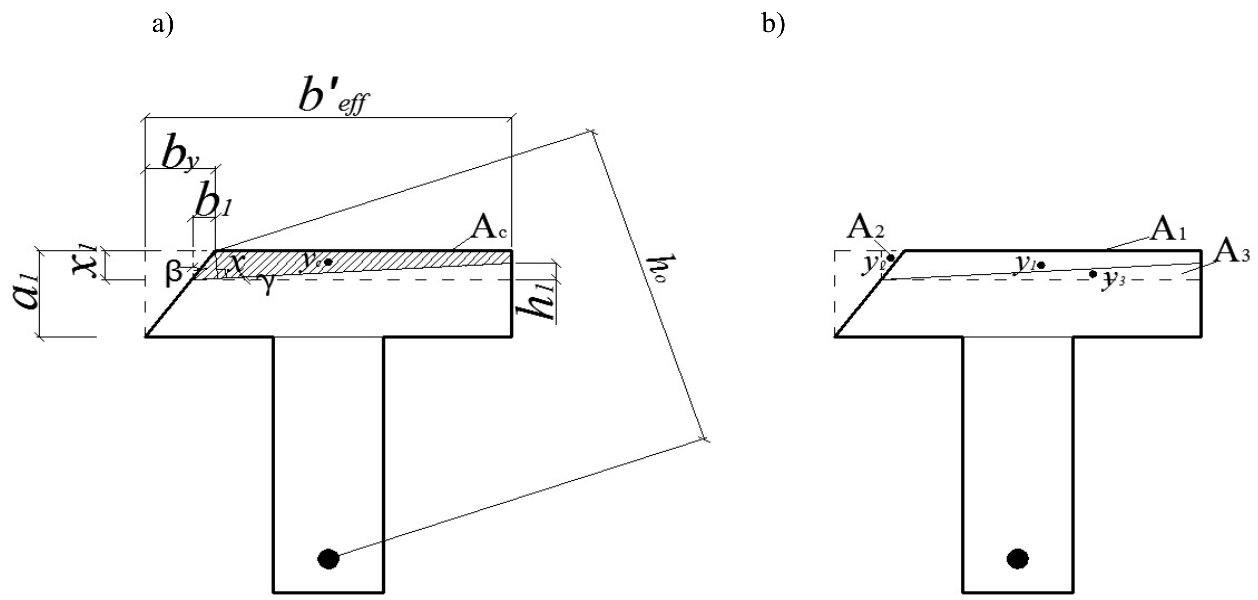


Fig. 5.9. Determination of the area of the compressed zone: a – an accepted outline of the compressed zone; b – a division of the compressed zone into simple figures.

Stresses σ_s in a stretched reinforcement should be defined according to the formula (4.7):

$$\sigma_{si} = \frac{\sigma_{sc,u}}{1 - \frac{\omega}{1.1}} \left(\frac{\omega}{\xi_i} - 1 \right),$$

where:

$$\omega = \alpha - 0.008 \times f_{cd} = 0.85 - 0.008 \times 14.5 = 0.734;$$

$$\sigma_{sc,u} = 400\ MPa.$$

The amount of the operating height h_0(4.28):

$$h_0 = \left(h - \left(a + \frac{d}{2} \right) + \left(\frac{b'_{eff}}{2} - b_y \right) \times \text{tg}\gamma \right) \times \cos\gamma$$

$$= (0.4 - \left(0.03 + \frac{0.02}{2} \right) + \left(\frac{0.4}{2} - 0.07813 \right) \times \text{tg}\gamma) \times \cos\gamma$$

$$= (0.36 + 0.1218 \times \text{tg}\gamma) \times \cos\gamma;$$

$$\xi_i = \frac{x_1}{h_o} = \frac{x_1}{(0.36 + 0.1218 \times \text{tg}\gamma) \times \cos\gamma}. \quad (5.64)$$

Coordinates of the center of mass of compressed concrete x_c, y_c can be expressed in the following way:

$$x_c = \frac{A_1 \times x_1 - A_2 \times x_2 - A_3 \times x_3}{A_1 - A_2 - A_3}; \quad (5.65)$$

$$y_c = \frac{A_1 \times y_1 - A_2 \times y_2 - \times \cdot y_3}{A_1 - A_2 - A_3}; \quad (5.66)$$

where (Fig. 5.9, b):

$$A_1 = x_1 \times \left(b'_{eff} - b_y + \frac{x_1 \times b_y}{a_1}\right); \quad (5.67)$$

$$A_2 = \frac{1}{2} x_1 \times \frac{x_1 \times b_y}{a_1} \quad ;(5.68)$$

$$A_3 = \frac{1}{2}\left(b'_{eff} - b_y + \frac{x_1 \cdot b_y}{a_1}\right) \cdot \left(b'_{eff} - b_y + \frac{x_1 \cdot b_y}{a_1}\right) \cdot tg\gamma; \ (5.69)$$

$$x_1 = \frac{b'_{eff} - b_2}{2} + b_2; \quad (5.70)$$

$$x_2 = b_2 + \frac{b_1}{3}; \quad (5.71)$$

$$x_3 = b'_{eff} - \frac{b'_{eff} - b_2}{3}; \quad (5.72)$$

$$y_1 = h - (a + \frac{d}{2}) - \frac{x_1}{2}; \quad (5.73)$$

$$y_2 = h - (a + \frac{d}{2}) - \frac{x_1}{3}; \quad (5.74)$$

$$y_3 = h - (a + \frac{d}{2}) - x_1 + \frac{\left(b'_{eff} - b_y + b_1\right) \times tg\gamma}{3}. \quad (5.75)$$

The third equation (4.19) expresses a static moment of the compressed zone of the concrete relatively to the axis *y* which can be defined by dividing into simple figures (Fig. 5.10).

$$b_2 = b_y - b_1 = a_1 \cdot tg\beta - \frac{x_1 \cdot b_y}{a_1}; \quad (5.76)$$

$$A_4 = A_6 - A_7 - A_8; \quad (5.77)$$

$$x_4 = \frac{A_6 \times x_6 - A_7 \times x_7 - A_8 \times x_8}{A_6 - A_7 - A_8}; \quad (5.78)$$

$$A_5 = A_9 - A_{10}; \quad (5.79)$$

$$x_5 = \frac{A_9 \times x_9 - A_{10} \times x_{10}}{A_9 - A_{10}}; \quad (5.80)$$

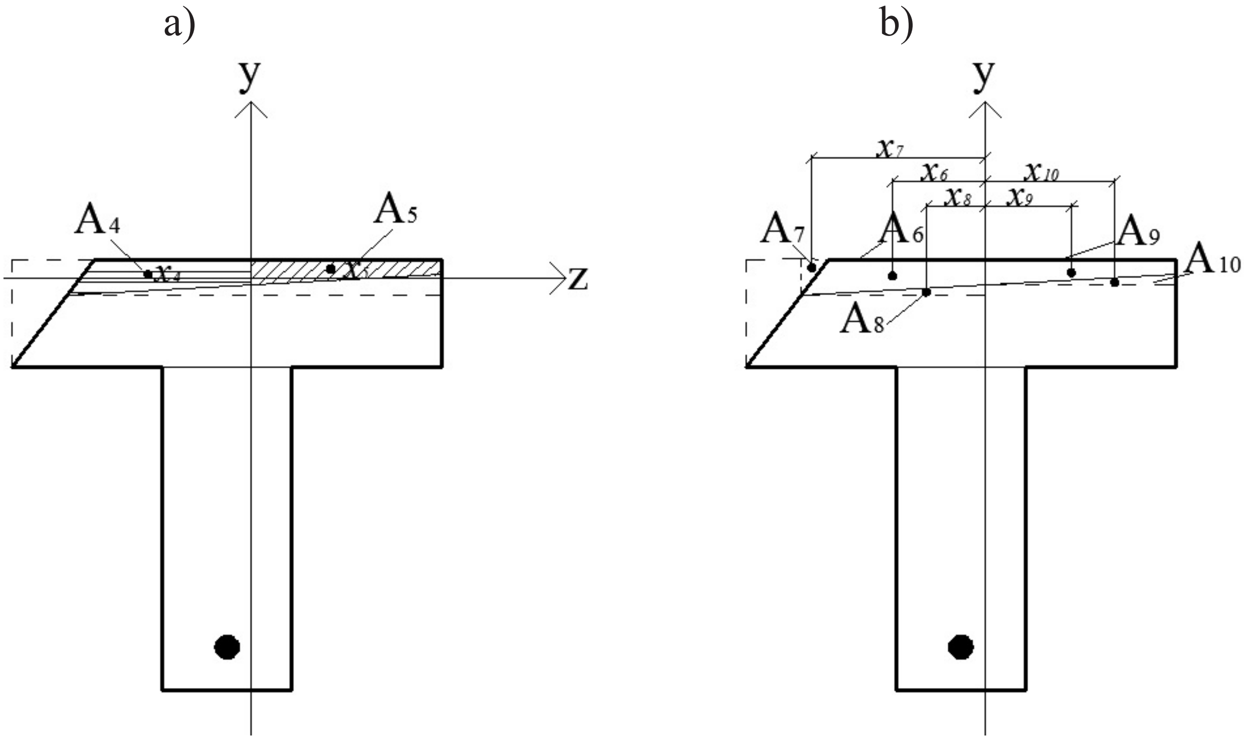


Fig. 5.10. A static moment relatively to the main axis *y*: a – a balance of the parts of a section relatively to the axis *y*; b – simple compound figures.

Where:

$$A_6 = x_1 \times (x_c \times b_2); \quad (5.81)$$

$$A_7 = \frac{1}{2} x_1 \times b_1 \quad ; \quad (5.82)$$

$$A_8 = \frac{1}{2} \times (x_c \times b_2) \times (x_c \times b_2) \times tg\gamma; \quad (5.83)$$

$$A_9 = (b'_{eff} - x_c) \times (x_1 - (x_c \times b_2) \times tg\gamma); \quad (5.84)$$

$$A_{10} = \frac{1}{2} \times (b'_{eff} - x_c) \times (b'_{eff} - x_c) \times tg\gamma; \quad (5.85)$$

$$x_6 = \frac{x_c - b_2}{2}; \quad (5.86)$$

$$x_7 = x_c - b_2 - \frac{b_1}{3}; \quad (5.87)$$

$$x_8 = \frac{x_c - b_2}{3}; \quad (5.88)$$

$$x_9 = \frac{x_c - b_2}{2}; \quad (5.89)$$

$$x_{10} = \frac{3 \times (x_c - b_2)}{2}. \quad (5.90)$$

After a substitution of all the quantities in the system of equations, the following expressions were received:

$$1).f_{cd}\times(x_1\times\left(b'_{eff}-b_y+\tfrac{x_1\times b_y}{a_1}\right)-\tfrac{1}{2}x_1\times\tfrac{x_1\times b_y}{a_1}\quad-\tfrac{1}{2}\left(b'_{eff}-b_y+\tfrac{x_1\times b_y}{a_1}\right)\times \tag{5.91}$$

$$\times\left(b'_{eff}-b_y+\frac{x_1\times b_y}{a_1}\right)\times tg\gamma)+\frac{\sigma_{sc,u}}{1-\frac{\omega}{1.1}}\left(\frac{\omega\times h_o}{x_1}-1\right)\times A_s=0;$$

$$2).\ M-f_{cd}\times(x_1\times\left(b'_{eff}-b_y+\tfrac{x_1\times b_y}{a_1}\right)-\tfrac{1}{2}x_1\times\tfrac{x_1\times b_y}{a_1}\quad-\tfrac{1}{2}\left(b'_{eff}-b_y+\tfrac{x_1\times b_y}{a_1}\right)\times$$

$$\times\left(b'_{eff}-b_y+\tfrac{x_1\times b_y}{a_1}\right)\times tg\gamma)\times \tag{5.92}$$

$$\times(x_1\left(b'_{eff}-b_y+\frac{x_1\times b_y}{a_1}\right)\left(h-\left(a+\frac{d}{2}\right)-\frac{x_1}{2}\right)-\frac{1}{2}x_1\times\frac{x_1\times b_y}{a_1}\left(h-\left(a+\frac{d}{2}\right)-\frac{x_1}{3}\right)-$$

$$-\frac{1}{2}\left(b'_{eff}-b_y+\frac{x_1\times b_y}{a_1}\right)\left(b'_{eff}-b_y+\frac{x_1\times b_y}{a_1}\right)\times tg\gamma\times(h-(a+\frac{d}{2})-x_1+$$

$$+\frac{\left(b'_{eff}-b_y+\frac{x_1\times b_y}{a_1}\right)\times tg\gamma}{3}))/(x_1\times\left(b'_{eff}-b_y+\frac{x_1\times b_y}{a_1}\right)-\frac{1}{2}x_1\times\frac{x_1\times b_y}{a_1}-$$

$$-\frac{1}{2}\left(b'_{eff}-b_y+\frac{x_1\times b_y}{a_1}\right)\left(b'_{eff}-b_y+\frac{x_1\times b_y}{a_1}\right)\times tg\gamma)=0;$$

$$3).\ (x_1\times(x_c\times b_2)-\tfrac{1}{2}x_1\times b_1-\tfrac{1}{2}\times(x_c\times b_2)\times(x_c\times b_2)\times tg\gamma)\times \tag{5.93}$$

$$\times\left(\frac{x_1(x_c\times b_2)\frac{x_c-b_2}{2}-\frac{1}{2}x_1\times b_1\times\left(x_c-b_2-\frac{b_1}{3}\right)-\frac{1}{2}(x_c\times b_2)(x_c\times b_2)\times tg\gamma\times\frac{x_c-b_2}{3}}{x_1\times(x_c\times b_2)-\frac{1}{2}x_1\times b_1-\frac{1}{2}\times(x_c\times b_2)\times(x_c\times b_2)\times tg\gamma}\right)-$$

$$-((b'_{eff}-x_c)\times(x_1-(x_c\times b_2)\times tg\gamma)-\frac{1}{2}\times(b'_{eff}-x_c)\times(b'_{eff}-x_c)\times tg\gamma)\times$$

$$\times\frac{(b'_{eff}-x_c)(x_1-(x_c\times b_2)\times tg\gamma)\frac{x_c-b_2}{2}-\frac{1}{2}(b'_{eff}-x_c)(b'_{eff}-x_c)\times tg\gamma\times\frac{3(x_c-b_2)}{2}}{(b'_{eff}-x_c)(x_1-(x_c\times b_2)\times tg\gamma)-\frac{1}{2}(b'_{eff}-x_c)(b'_{eff}-x_c)\times tg\gamma}=0.$$

In numerical form:

$$1).\ 14.5\times(x_1\times\left(0.4-0.078+\tfrac{x_1\times 0.078}{0.1}\right)-\tfrac{1}{2}x_1\times\tfrac{x_1\times 0.078}{0.1}\quad-\tfrac{1}{2}\left(0.4-0.078+\tfrac{x_1\times 0.078}{0.1}\right)\times$$

$$\times\left(0.4-0.078+\frac{x_1\times 0.078}{0.1}\right)\times tg\gamma)+\frac{400}{1-\frac{0.734}{1.1}}\times$$

$$\times\left(\frac{0.734\times((0.36+0.1218\times \mathrm{tg}\gamma)\times\cos\gamma)}{x_1}-1\right)\times 0.000314=0;$$

2). $M-14.5(x_1\times\left(0.4-0.078+\frac{x_1\times 0.078}{0.1}\right)-\frac{1}{2}x_1\frac{x_1\times 0.078}{0.1}\quad-\frac{1}{2}\left(0.4-0.078+\frac{x_1\times 0.078}{0.1}\right)\times$
$\left(0.4-0.078+\frac{x_1\times 0.078}{0.1}\right)\times tg\gamma)\times$

$$\times(x_1\times\left(0.4-0.078+\frac{x_1\times 0.078}{0.1}\right)\times\left(0.4-\left(0.03+\frac{0.02}{2}\right)-\frac{x_1}{2}\right)-\frac{1}{2}x_1\times\frac{x_1\times 0.078}{0.1}\times$$
$$\times\left(0.4-\left(0.03+\frac{0.02}{2}\right)-\frac{x_1}{3}\right)-$$

$$-\frac{1}{2}\left(0.4-0.078+\frac{x_1\times 0.078}{0.1}\right)\left(0.4-0.078+\frac{x_1\times 0.078}{0.1}\right)\times tg\gamma\times(0.4-(0.03+\frac{0.02}{2})-x_1+$$

$$+\frac{\left(0.4-0.078+\frac{x_1\times 0.078}{0.1}\right)\times tg\gamma}{3}))/(x_1\left(0.4-0.078+\frac{x_1\times 0.078}{0.1}\right)-\frac{1}{2}x_1\frac{x_1\times 0.078}{0.1}-$$

$$-\frac{1}{2}\left(0.4-0.078+\frac{x_1\times 0.078}{0.1}\right)\left(0.4-0.078+\frac{x_1\times 0.078}{0.1}\right)\times tg\gamma)=0;$$

3). $(x_1\times\left(x_c\times\left(0.078-\frac{x_1\times 0.078}{0.1}\right)\right)-\frac{1}{2}x_1\times\frac{x_1\times 0.078}{0.1}-\frac{1}{2}\times\left(x_c\times\left(0.078-\frac{x_1\times 0.078}{0.1}\right)\right)\times$

$$\times\left(x_c\times\left(0.078-\frac{x_1\times 0.078}{0.1}\right)\right)\times tg\gamma)\times$$

$$\times(x_1\times\left(x_c\times\left(0.078-\frac{x_1\times 0.078}{0.1}\right)\right)\frac{x_c-\left(0.078-\frac{x_1\times 0.078}{0.1}\right)}{2}-\frac{1}{2}x_1\times\frac{x_1\times 0.078}{0.1}\times$$

$$\times\left(x_c-\left(0.078-\frac{x_1\times 0.078}{0.1}\right)-\frac{(\frac{x_1\times 0.078}{0.1})}{3}\right)-\frac{1}{2}\times\left(x_c\times\left(0.078-\frac{x_1\times 0.078}{0.1}\right)\right)\times$$

$$\times\left(x_c\times\left(0.078-\frac{x_1\times 0.078}{0.1}\right)\right)\times tg\gamma\times\frac{x_c-\left(0.078-\frac{x_1\times 0.078}{0.1}\right)}{3})/(x_1\times$$

$$\times\left(x_c\times\left(0.078-\frac{x_1\times 0.078}{0.1}\right)\right)-\frac{1}{2}x_1\times\frac{x_1\times 0.078}{0.1}-\frac{1}{2}\left(x_c\times\left(0.078-\frac{x_1\times 0.078}{0.1}\right)\right)\times$$

$$\times\left(x_c\times\left(0.078-\frac{x_1\times 0.078}{0.1}\right)\right)\times tg\gamma)-$$

$$-((0.4\quad-x_c)\times\left(x_1-\left(x_c\times\left(0.078-\frac{x_1\times 0.078}{0.1}\right)\right)\times tg\gamma\right)-\frac{1}{2}\times(0.4-x_c)(0.4-x_c)\times tg\gamma)\ \times$$

$$\times ((0.4 - x_c) \times \left(x_1 - \left(x_c \times \left(0.078 - \frac{x_1 \times 0.078}{0.1}\right)\right) \times tg\gamma\right) \times \frac{x_c - \left(0.078 - \frac{x_1 \times 0.078}{0.1}\right)}{2} -$$

$$-\frac{1}{2} \times (0.4 - x_c) \times (0.4 - x_c) \times tg\gamma \times \frac{3 \times (x_c - \left(0.078 - \frac{x_1 \times 0.078}{0.1}\right))}{2})/$$

$$/((0.4 - x_c) \times \left(x_1 - \left(x_c \times \left(0.078 - \frac{x_1 \times 0.078}{0.1}\right)\right) \times tg\gamma\right) - \frac{1}{2}(0.4 - x_c)(0.4 - x_c) \times tg\gamma) = 0.$$

In the third equation of the equality of static moments x_c is a coordinate of the center of mass in numerical form:

$$x_c = (x_1 \times \left(0.4 - 0.078 + \frac{x_1 \times 0.078}{0.1}\right)\left(\frac{0.4 - \left(0.078 - \frac{x_1 \times 0.078}{0.1}\right)}{2} + \left(0.078 - \frac{x_1 \times 0.078}{0.1}\right)\right) -$$

$$-\frac{1}{2} x_1 \times \frac{x_1 \times \left(0.078 - \frac{x_1 \times 0.078}{0.1}\right)}{0.1} \times \left(\left(0 \times 0.078 - \frac{x_1 \times 0.078}{0.1}\right) + \frac{\left(\frac{x_1 \times 0.078}{0.1}\right)}{3}\right) -$$

$$-\frac{1}{2}\left(0.4 - 0.078 + \frac{x_1 \times 0.078}{0.1}\right) \times \left(0.4 - \ 0.078 + \frac{x_1 \times \ 0.078}{0.1}\right) \times tg\gamma \times (0.4 -$$

$$-\frac{0.4 - \left(0.078 - \frac{x_1 \times 0.078}{0.1}\right)}{3}))/(x_1 \times \left(0.4 - 0.078 + \frac{x_1 \times 0.078}{0.1}\right) -$$

$$-\frac{1}{2} x_1 \times \frac{x_1 \times 0.078}{0.1} \quad -\frac{1}{2}\left(0.4 - 0.078 + \frac{x_1 \times 0.078}{0.1}\right) \times \left(0.4 - 0.078 + \frac{x_1 \times 0.078}{0.1}\right) \times tg\gamma).$$

The solution of this system gives the following roots:

$$\begin{cases} x_1 = 0.073 \text{ m}; \\ \text{M} = 0.058 \text{ MNm}; \\ \gamma = 0.089 \text{ rad.} \end{cases}$$

At this value of x the stretching stresses in a reinforcing bar will be equal to $\sigma_s = 3149\ MPa \gg f_{yd} = 365\ MPa$according to the formula (4.7).

After the recalculation of the system taking that$\sigma_s = f_{yd} = 365\ MPa$ the following equations were obtained:

$$\begin{cases} f_{cd} \times A_c + f_{yd} \times A_s = 0; \\ M - f_{cd} \times A_c \times y_c = 0; \\ S_y = A_4 x_4 - A_5 x_5. \end{cases}$$

As a result, the following roots were obtained:

$$\begin{cases} x_1 = 0.031 \text{ m}; \\ \text{M} = 0.03807 \text{ MNm} = 38.07 \text{ kNm}; \\ \gamma = 0.054 \text{ rad} \approx 3°. \end{cases}$$

The calculations that were performed using the method from this thesis, allowed proving the possibility of reinforcing this type of damaged beams. After the reinforcement had been done, the monitoring of the object lasted for a year, which showed that there were no defects or other deviations from normal operating parameters of the construction. Therefore, the bearing capacity of the damaged construction was determined reliable and secure.

5.2.2. The implementation in the educational process. The results obtained in this thesisresearch are used in the educational process in the Odessa State Academy of Construction and Architecture at lectures and practical classes on the discipline “Building constructions”, which is taught to students of the course “Building” (specialty “Urban Construction and Economy” and “Production of building materials, products and constructions” (the 3rd year of the Building Technology Institute of the Academy)). Conducting of these classes is performed at the Department of building constructions.

5.3. Conclusions for the Chapter

1. The residual bearing capacity of experimental samples was determined using the method of calculation of damaged bent reinforced concrete T-shaped elements which was developed in this thesis. The detailed example calculation of one of the proven beams, namely B 13, was done. The results of the calculation attest a sufficiently good agreement with the quantities, which were obtained during the natural experiment.

2. The research results were appliedduring the reconstruction of the industrial building of“Glavstroy” LLC in Odessa. The calculations that were performed using the method from this thesis, allowed validating the possibility of reinforcing such kind of a damaged bearing element. After the reinforcement had been done, the monitoring of the object lasted for a year, which showed that there were no defects or other deviations from normal operating parameters of the construction. Therefore, the bearing capacity of the damaged construction was determined reliable and secure enough.

3. The research results obtained in this thesisare used in the educational process in the Odessa State Academy of Construction and Architecture at lectures and practical classes on the discipline “Building constructions”.

GENERAL CONCLUSIONS

1. In operating practice cases of a damage of bent concrete elements often occur. However, despite a rather extensive research of damaged constructions, there are no recommendations for the assessment of residual bearing capacity in normative documents, which makes impossible further safe exploitation of buildings and constructions.

2. The monograph contains experimental data on the operation of the damaged beams of the T-shaped profile. 15 reinforced concrete experimental samples with damages were manufactured and tested. After analyzing the works and publications on the subject of research the following factors of variation were selected for the research: an angle of a damage ($\beta/90^0$), a damage depth ($a_1/h_f^/$) and a damaged part of a shelf (b_{eff1}/b_{eff2}). To analyze the influence of the selected factors of variation the experimental and statistical modeling was performed on the PC COMPEX.

It was established that the factor having the greatest impact on the bearing capacity of the beams, is the depth of the damage. Also the patterns between the combined effects of factors were revealed.

3. The method of manufacturing samples with a damage which is the closest to the real damage produced during the impact of various factors on the beams was developed. The proposed testing method made possible to study the stress-deformed condition of the damaged T-shaped beams in the zone of maximum stress, to determine the bearing capacity of these samples empirically. Certain patterns were seen during testing prototypes. Two main typical cases of destruction of prototypes (for reinforced and non-reinforced elements) were described.

4. The numerical experiment in PK LIRA 9.6 was fulfilled, based on the finite element method using of the nodes of the calculated scheme of the experimental samples as the primary unknown displacements and rotations. In general, the results of the calculation in the software package are sufficiently accurate and suitable for the preliminary

determination of the bearing capacity and prediction the natureof the destruction of the samples.

5. A method of calculation which takes into account the condition of the parallelism of force planes was developed: the resultant of the compressive forces in concrete, reinforcement and the resultant of the forces in the stretched reinforcement must lie in a plane that is parallel with the force plane (or coincide with it). The main design cases were allocated and their differences were described. The algorithm of calculation of damaged bent and oblique bent T-shaped beams was created; it is possible to determine the residual bearing capacity of the damaged element according to this algorithm.

6. The calculation by the developed method showed a good agreement with the experimental values. The coefficient of variation of the deviation of the observation values of the residual bearing capacity from the defined by the proposed method amounted to 13.82 %. Such metric indicates the possibility of the application of the method during the reconstruction of the detached constructive elements of buildings and constructions.

7. The results of the research have been introduced into the practice of design and building and into the educational process of the higher education institution for the training of students of building specialties.

APPENDIX

APPENDIX A

The distribution of longitudinal strains in concrete and reinforcement

Series 1 – samples B 3, B 6, B 12, B 13

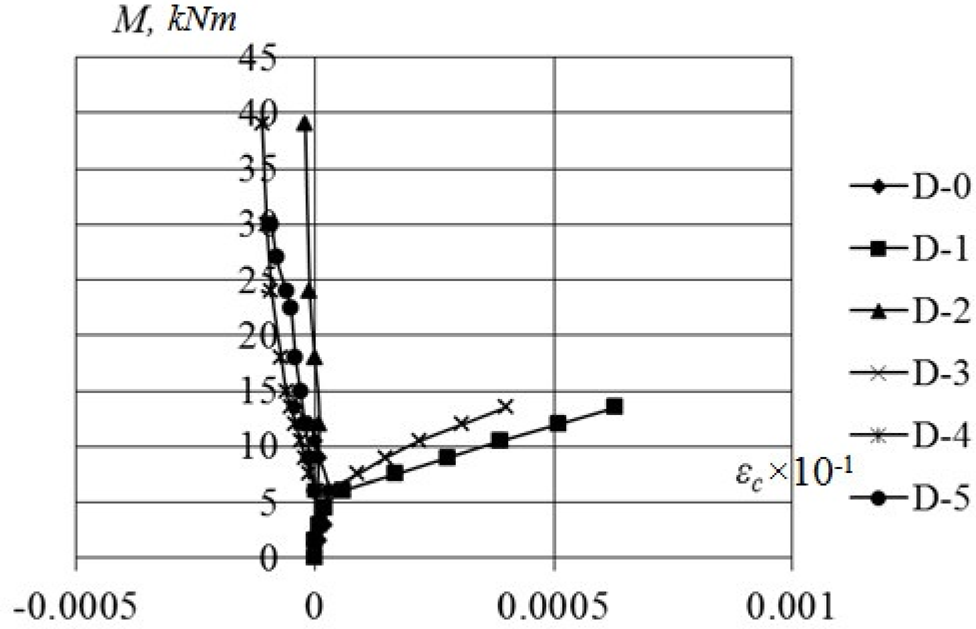


Fig. A. 1. The distribution of the longitudinal deformation of the beam B 3 by the length of the sample.

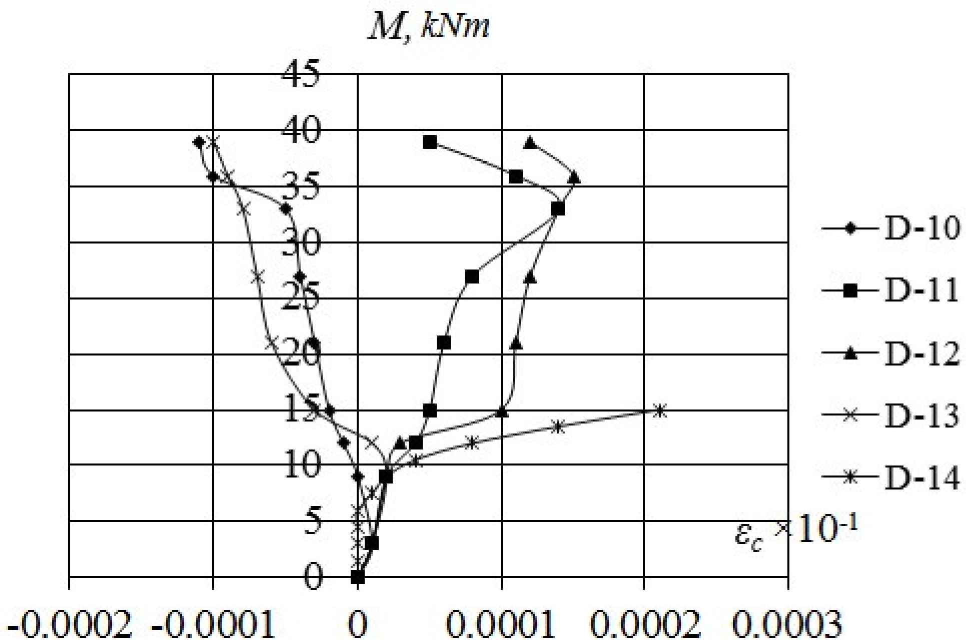


Fig. A. 2. The distribution of the longitudinal deformation of the beam B 3 by the height of the sample.

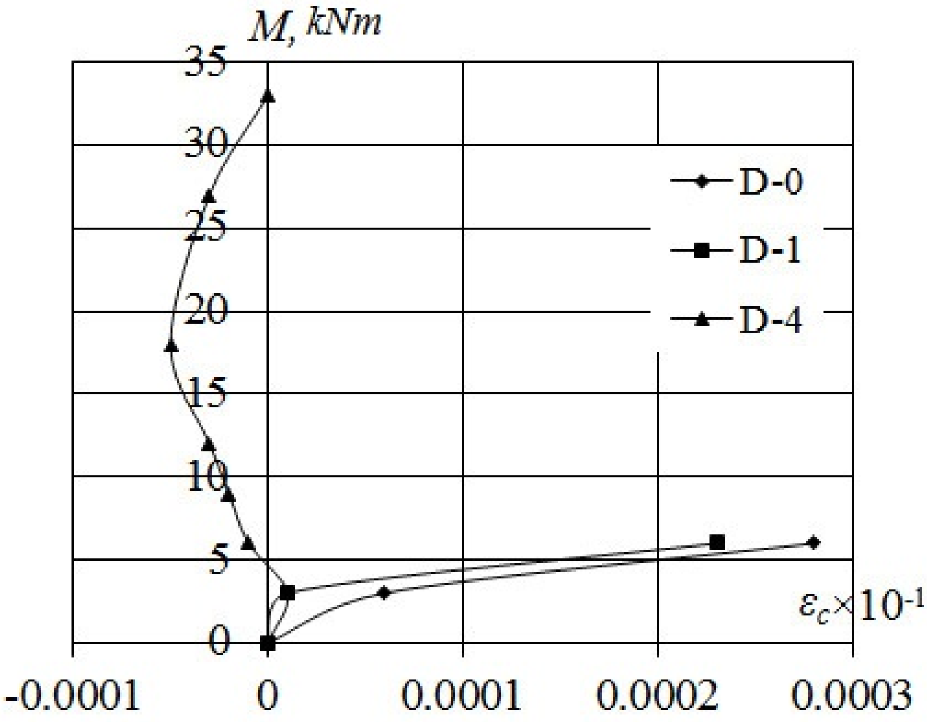


Fig. A. 3. The distribution of the longitudinal deformation of the beam B 6 by the length of the sample.

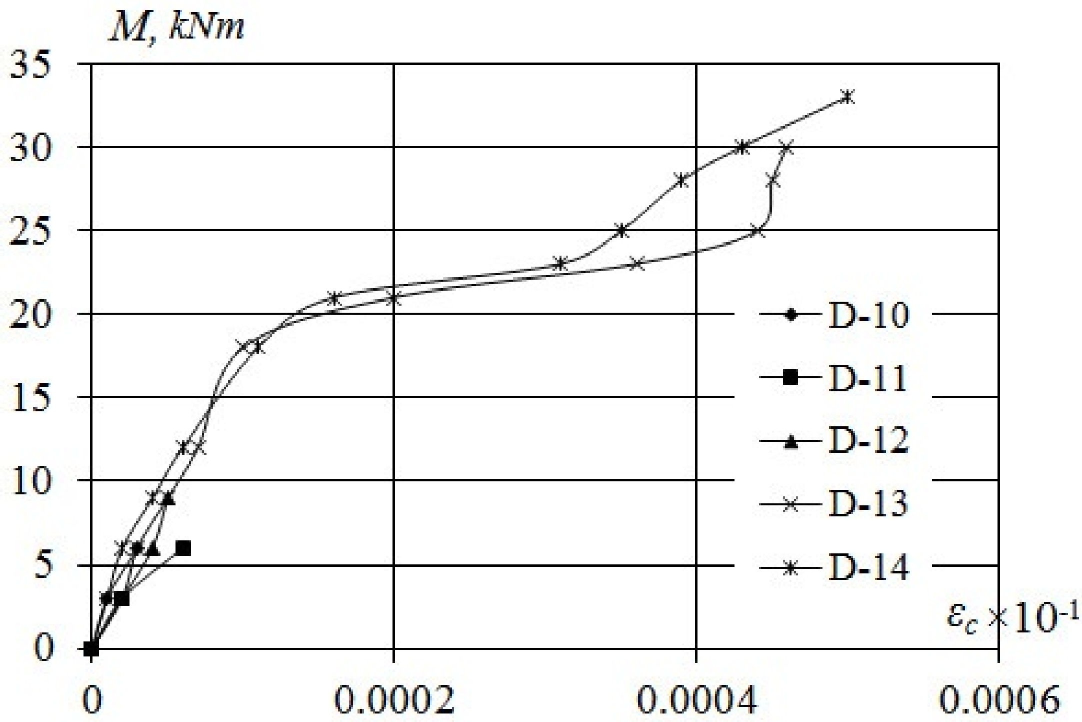


Fig. A. 4. The distribution of the longitudinal deformation of the beam B 6 by the height of the sample.

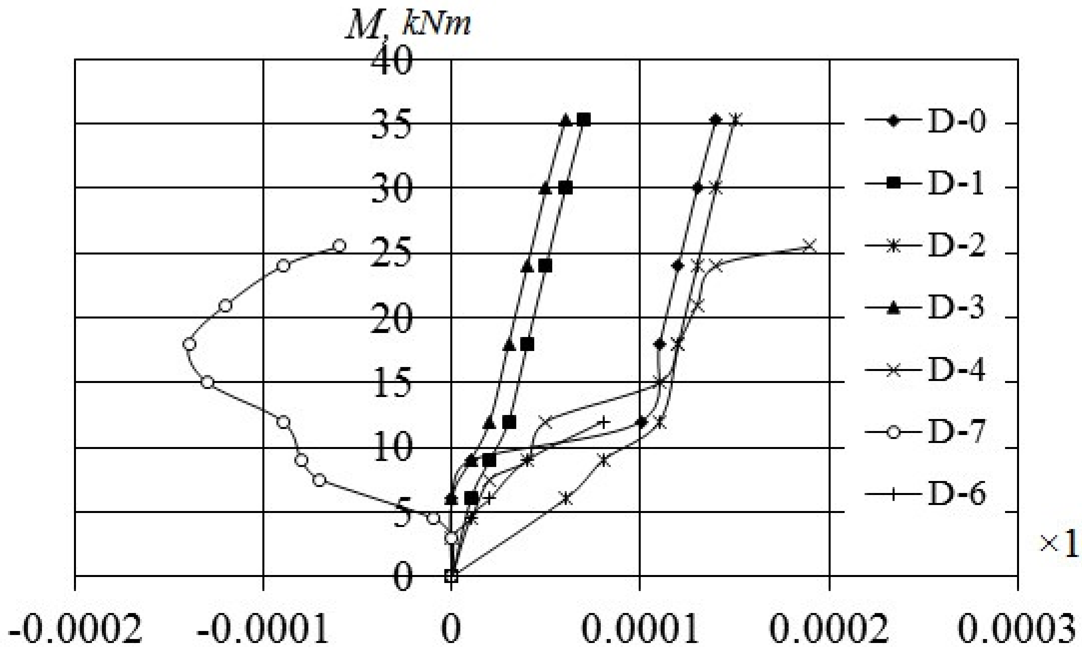


Fig. A. 5. The distribution of the longitudinal deformation of the beam B 12 by the length of the sample.

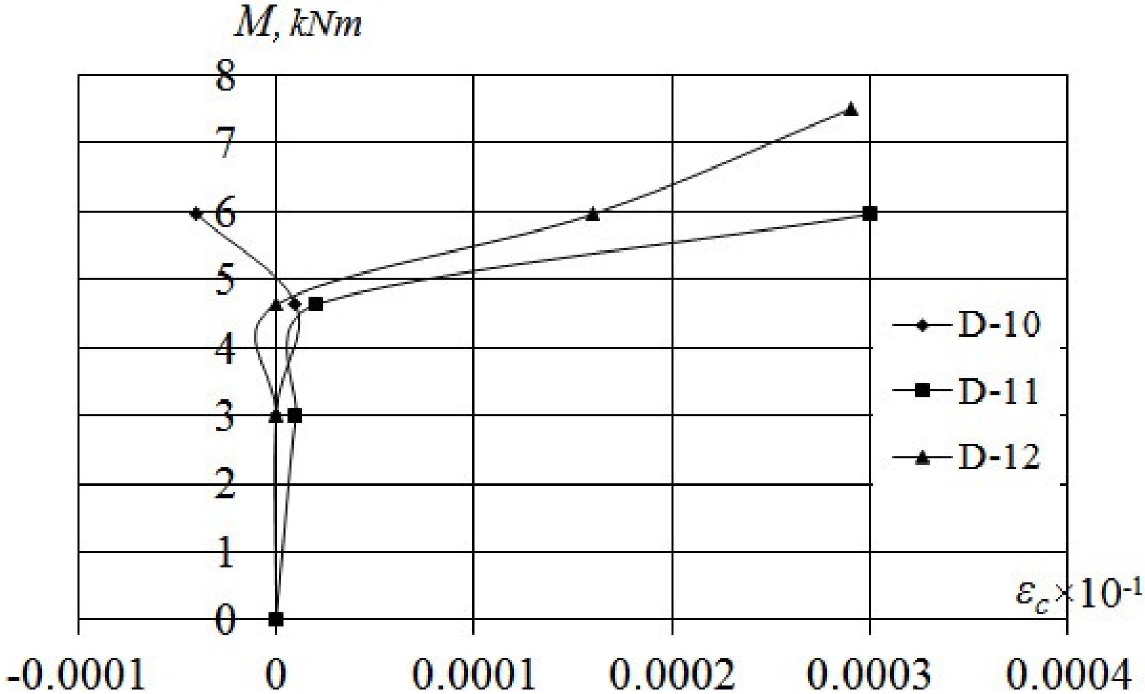


Fig. A. 6. The distribution of the longitudinal deformation of the beam B 12 by the height of the sample.

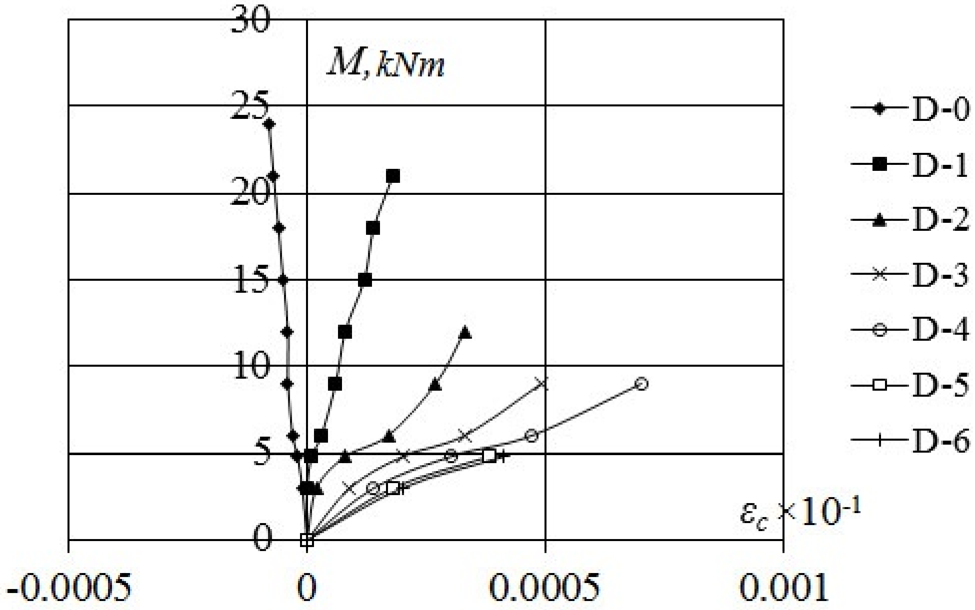


Fig. A. 7. The distribution of the longitudinal deformation of the beam B 13 by the length of the sample.

Series 2 – B 4, B 7, B 10, B 11, B 14, B 15

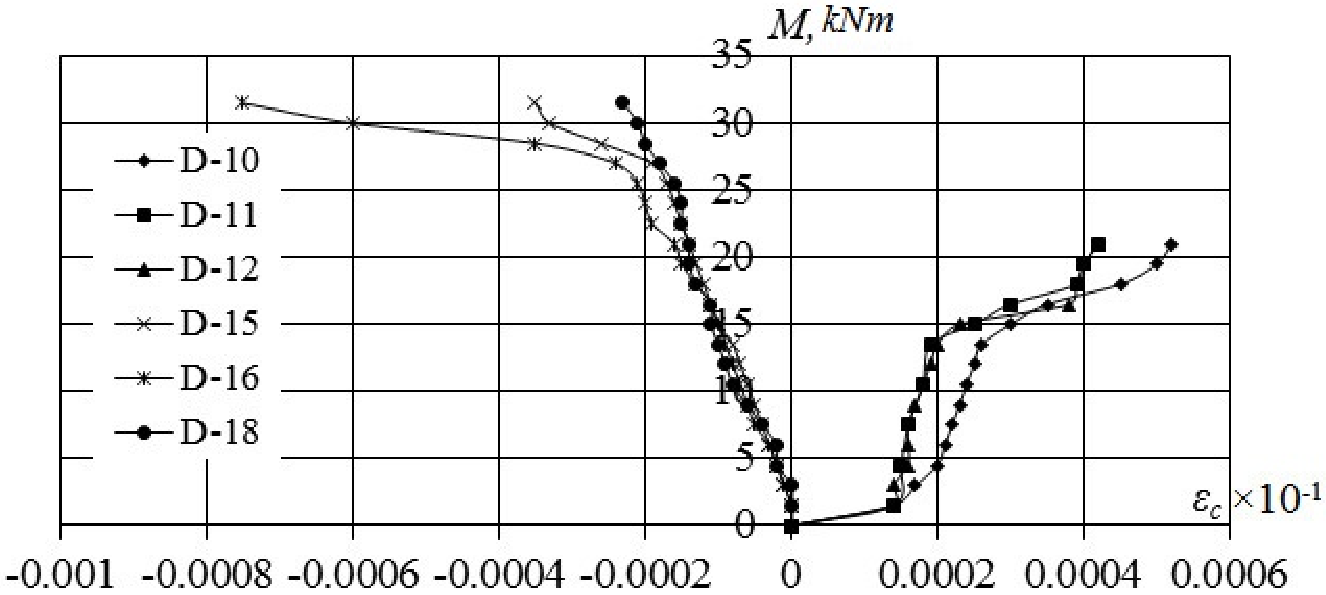


Fig. A. 8. The distribution of the longitudinal deformation of the beam B 4.

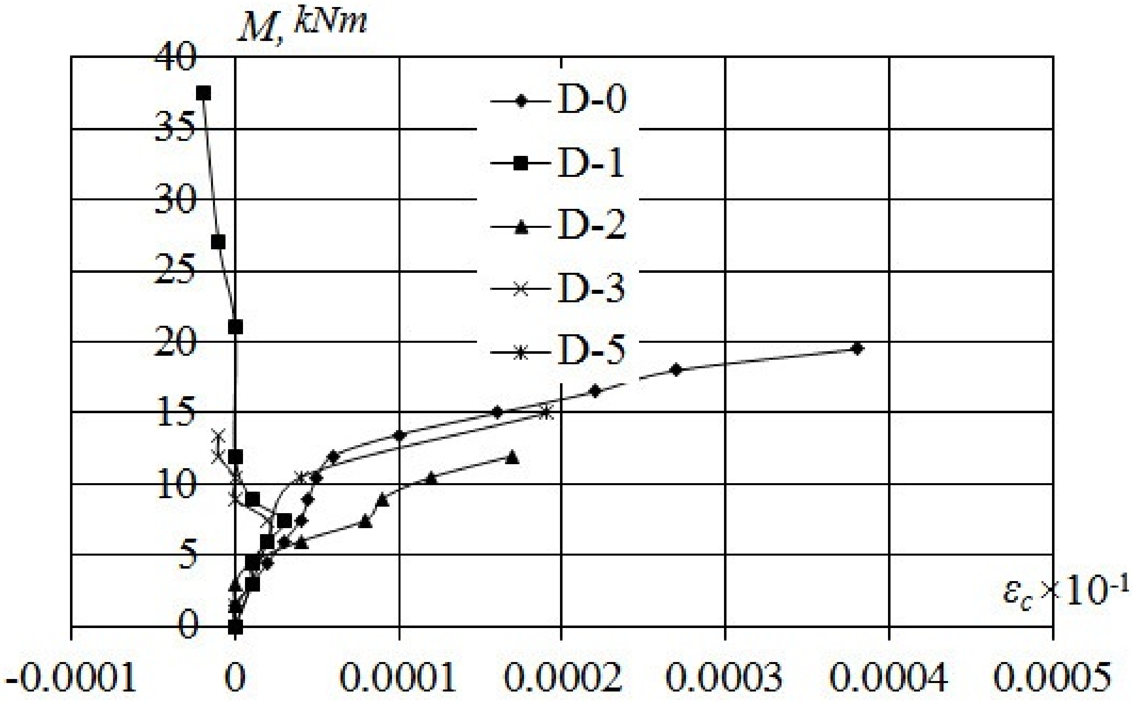


Fig. A. 9. The distribution of the longitudinal deformation of the beam B 7 by the length of the sample.

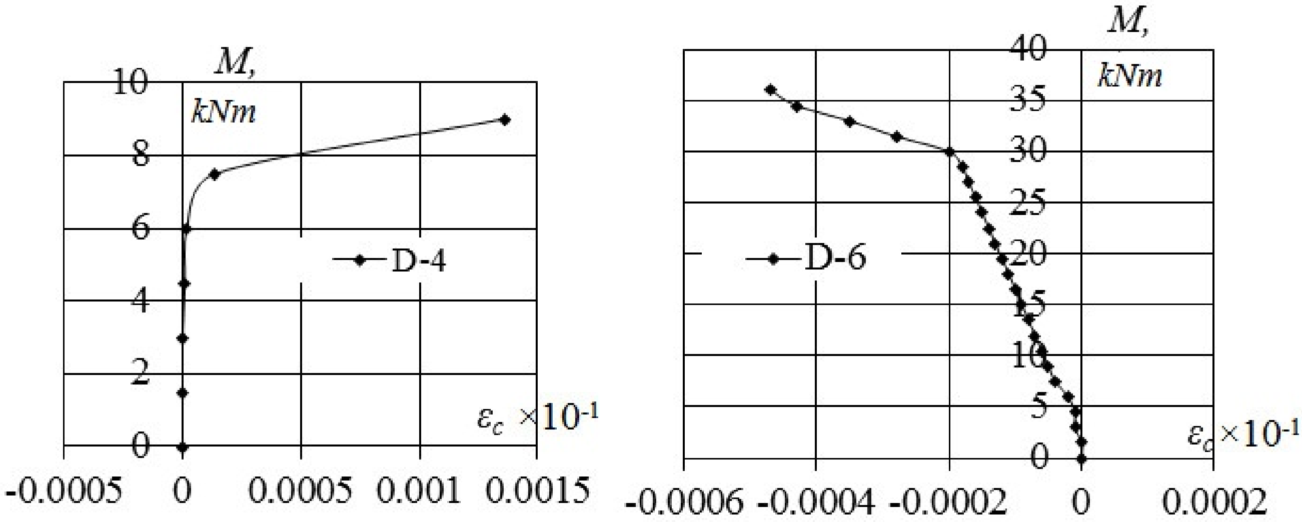


Fig. A. 10. The distribution of the longitudinal deformations of the beam B 7.

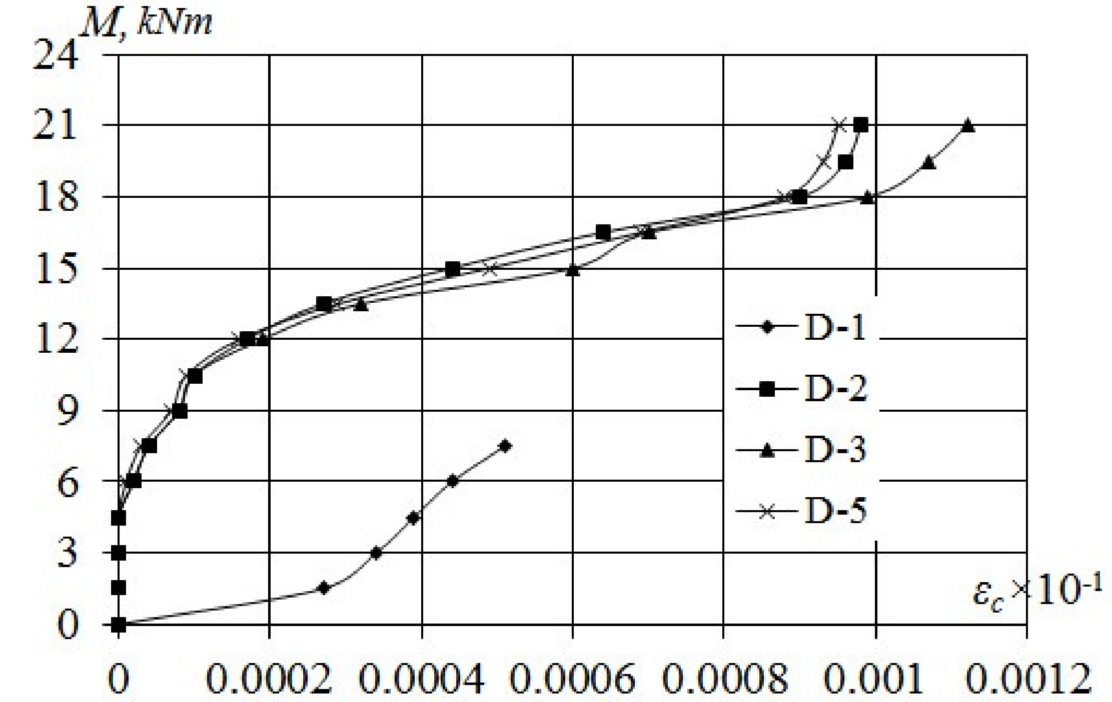


Fig. A. 11. The distribution of the longitudinal deformation of the beam B 11 by the length of the sample.

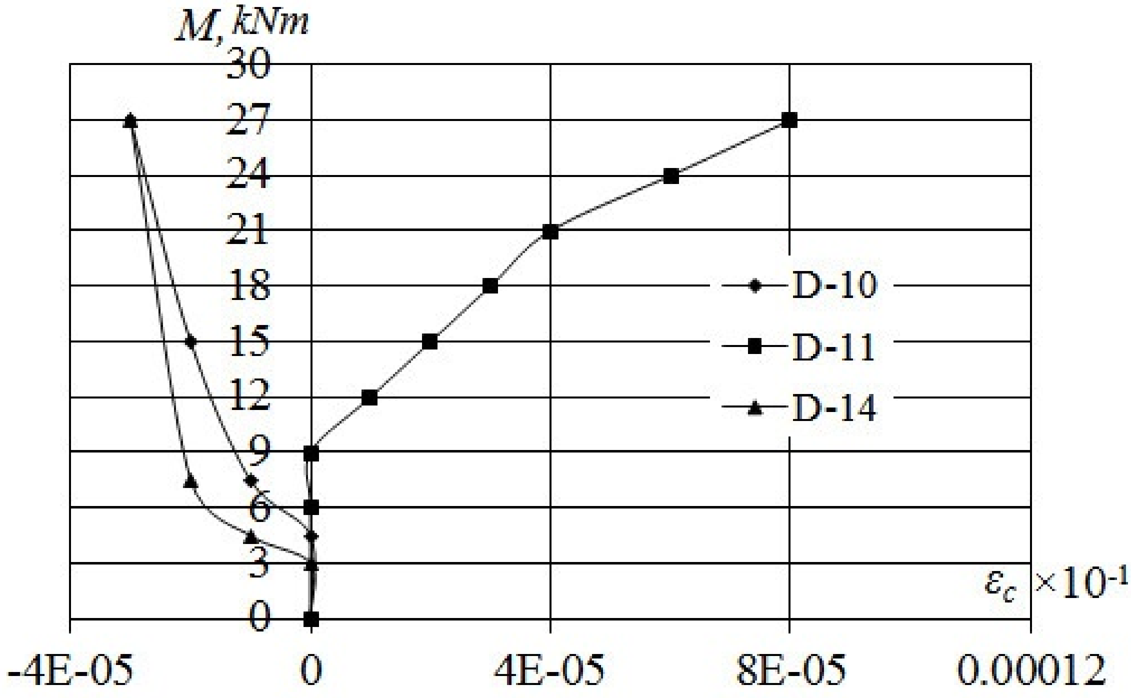


Fig. A. 12. The distribution of the longitudinal deformation of the beam B 11 by the height of the sample.

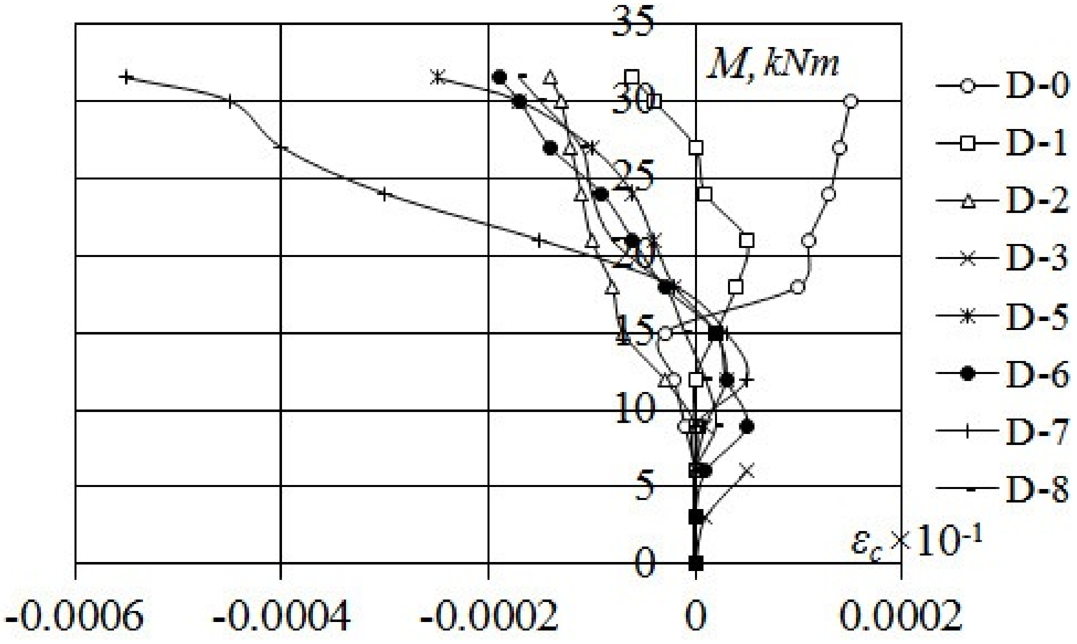


Fig. A. 13. The distribution of the longitudinal deformation of the beam B 14 by the length of the sample.

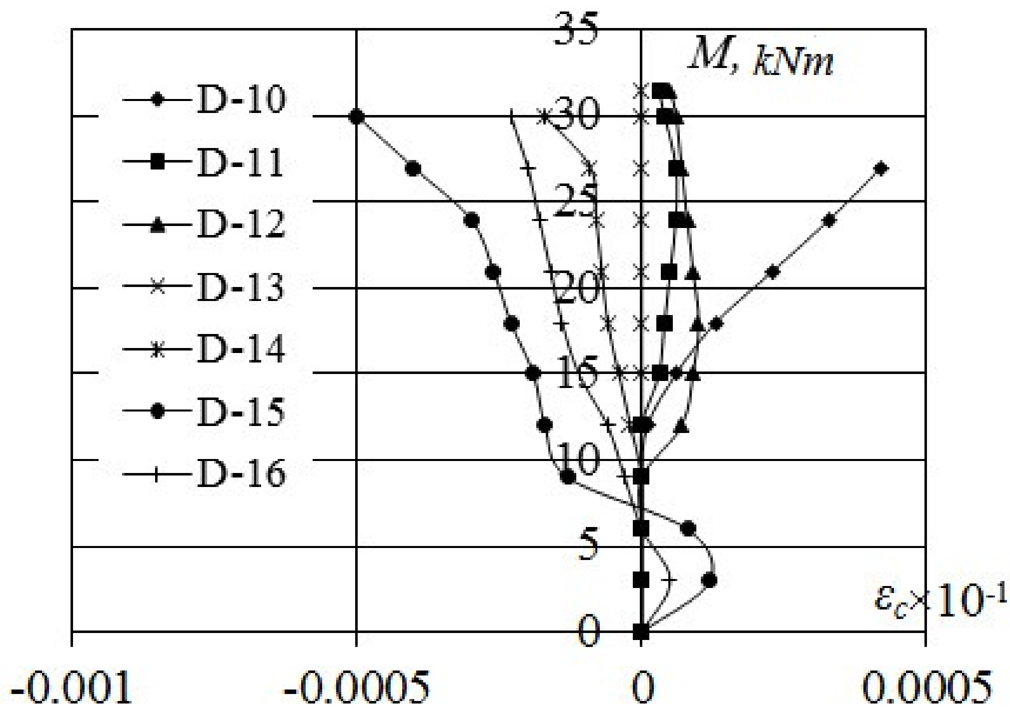


Fig. A. 14. The distribution of the longitudinal deformation of the beam B 14 by the height of the sample.

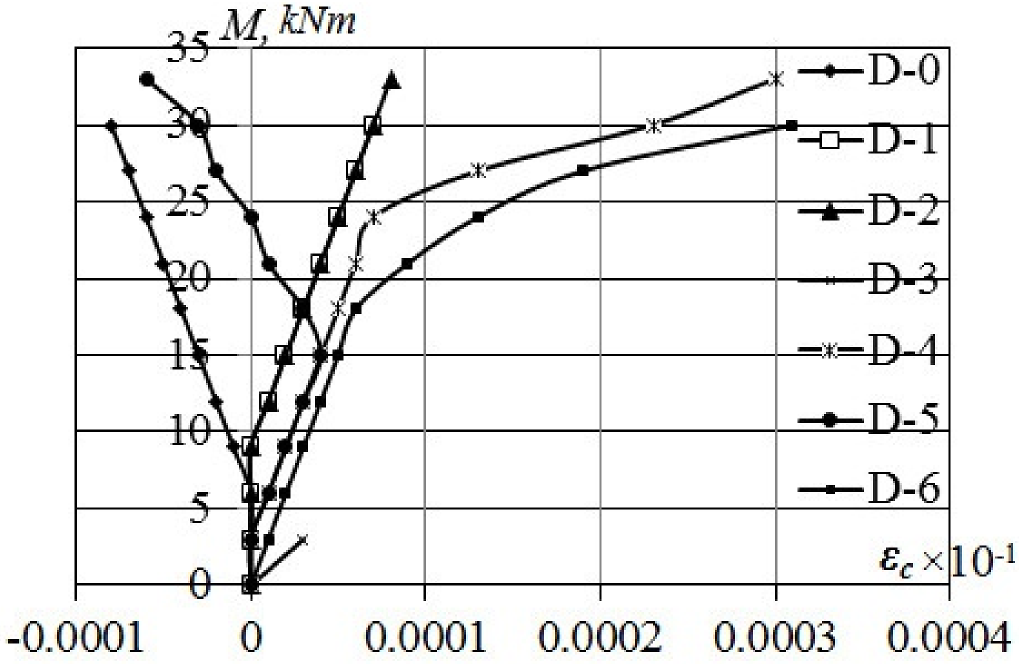


Fig. A. 15. The distribution of the longitudinal deformation of the beam B 15 by the length of the sample.

Series 3 – B 1, B 9

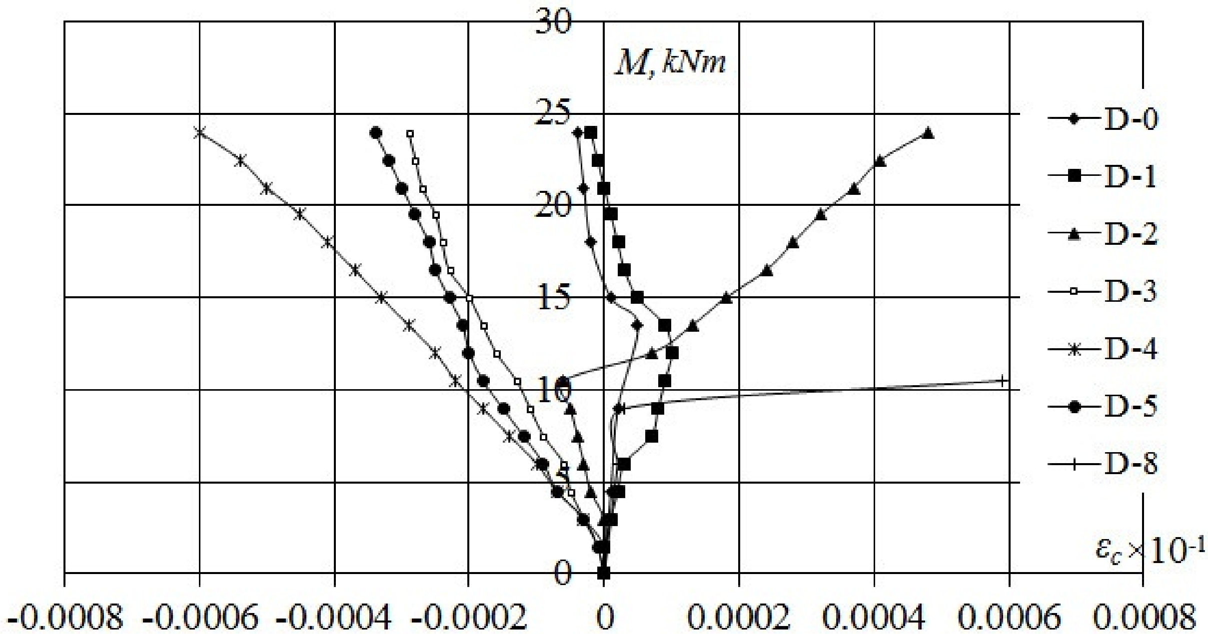


Fig. A. 16. The distribution of the longitudinal deformation of the beam B 1 by the length of the sample.

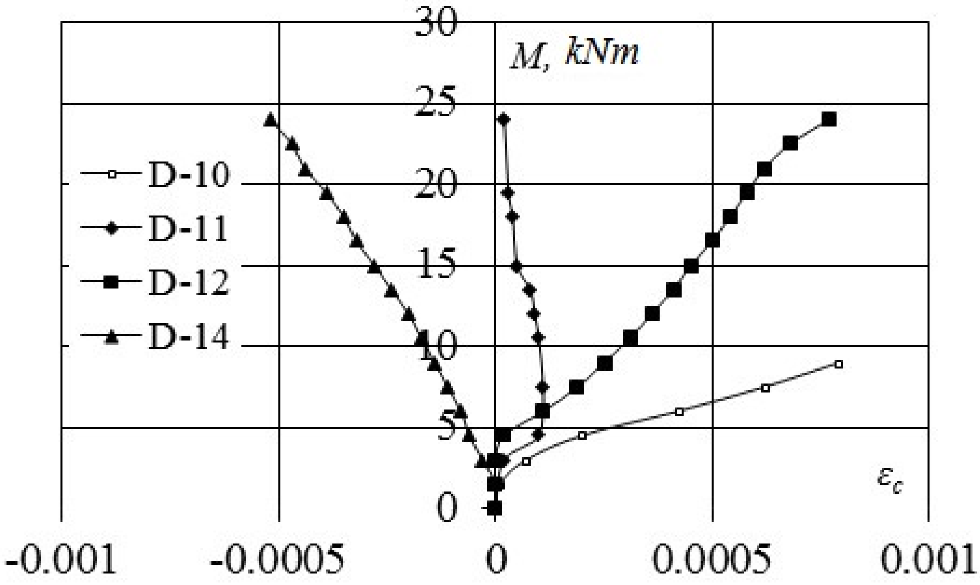


Fig. A. 17. The distribution of the longitudinal deformation of the beam B 1 by the height of the sample.

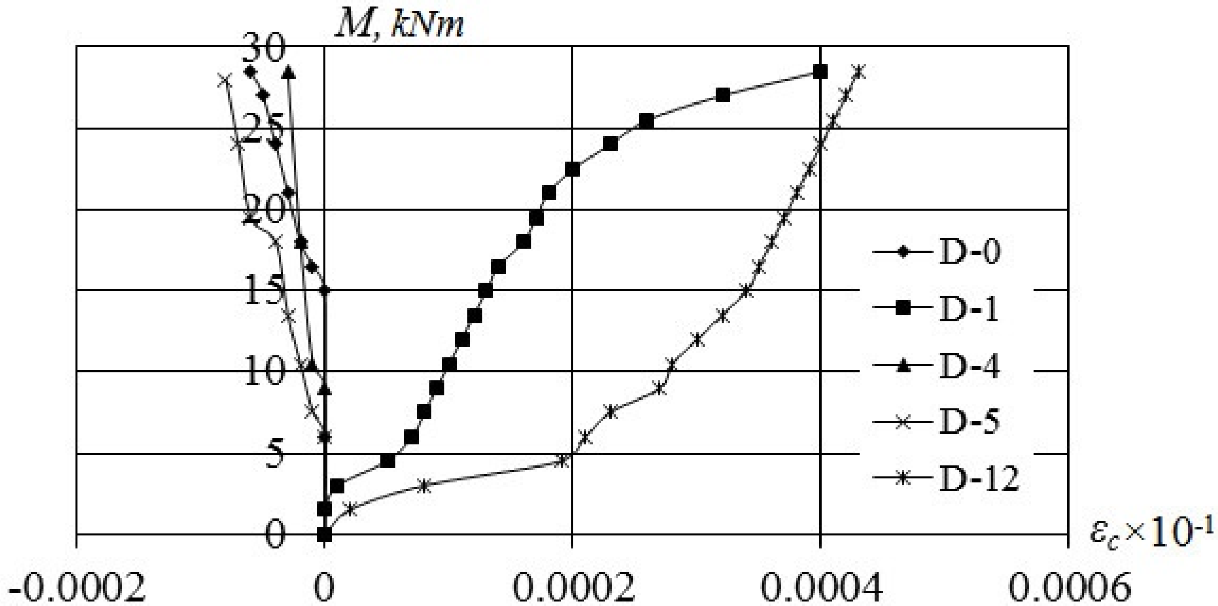


Fig. A. 18. The distribution of the longitudinal deformation of the beam B 9.

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