

ENGINEER



international scientific journal

ISSUE 3, 2026 Vol. 4

E-ISSN

3030-3893

ISSN

3060-5172



SLIB.UZ
Scientific library of Uzbekistan



A bridge between science and innovation



**TOSHKENT DAVLAT
TRANSPORT UNIVERSITETI**

Tashkent state
transport university



ENGINEER

A bridge between science and innovation

E-ISSN: 3030-3893

ISSN: 3060-5172

VOLUME 4, ISSUE 3

SEPTEMBER, 2026



engineer.tstu.uz

TASHKENT STATE TRANSPORT UNIVERSITY

ENGINEER

INTERNATIONAL SCIENTIFIC JOURNAL
VOLUME 4, ISSUE 3 SEPTEMBER, 2026

EDITOR-IN-CHIEF

SAID S. SHAUMAROV

Professor, Doctor of Sciences in Technics, Tashkent State Transport University

Deputy Chief Editor

Miraziz M. Talipov

Doctor of Philosophy in Technical Sciences, Tashkent State Transport University

Founder of the international scientific journal “Engineer” – Tashkent State Transport University, 100167, Republic of Uzbekistan, Tashkent, Temiryo‘lchilar str., 1, office: 465, e-mail: publication@tstu.uz.

The “Engineer” publishes the most significant results of scientific and applied research carried out in universities of transport profile, as well as other higher educational institutions, research institutes, and centers of the Republic of Uzbekistan and foreign countries.

The journal is published 4 times a year and contains publications in the following main areas:

- Engineering;
- General Engineering;
- Aerospace Engineering;
- Automotive Engineering;
- Civil and Structural Engineering;
- Computational Mechanics;
- Control and Systems Engineering;
- Electrical and Electronic Engineering;
- Industrial and Manufacturing Engineering;
- Mechanical Engineering;
- Mechanics of Materials;
- Safety, Risk, Reliability and Quality;
- Media Technology;
- Building and Construction;
- Architecture.

Tashkent State Transport University had the opportunity to publish the international scientific journal “Engineer” based on the **Certificate No. 1183** of the Information and Mass Communications Agency under the Administration of the President of the Republic of Uzbekistan. **E-ISSN: 3030-3893, ISSN: 3060-5172.** Articles in the journal are published in English language.

3	
engineer.tstu.uz	A bridge between science and innovation

Precast–monolithic frame building system based on permanent reinforced concrete shell formwork: Analysis of world practice and experimental substantiation

U.D. Khamrokulov¹^a

¹Samarkand State University of technical named after Mirzo Ulugbek, Samarkand, Uzbekistan

Abstract:

The paper examines the structural solutions of frame buildings erected by precast–monolithic technology in two consecutive stages — a systematic analysis of world practice and the author's own experimental verification. The objective is to identify the structural limits of the existing precast–monolithic frame systems, to substantiate the research gap they leave uncovered, and to verify experimentally a solution that fills this gap. More than seventeen foreign and CIS systems (KUB-2.5, ARKOS, RECON, UDS, Deltabeam, IMS, Structurapid, Thomas, RPS, BGK, Filigran and others) were compared against five criteria: the column solution, the beam solution, the floor solution, the detailing of the joint zone, and the limitation reported in the sources. The proposed solution was then tested under static loads on three types of two-bay, two-storey frame models designed at 1:4 scale by similarity theory. Four development directions of precast–monolithic technology were distinguished, and it was shown that none of them extends the principle of load-bearing formwork to the column cross-section. The ultimate load of the proposed solution exceeded that of the monolithic reference by 2.8 % (134.12 kN against 130.50 kN) and the load at limiting deflection by 3.0 %, while the difference between the longitudinal strains of the shell and the core did not exceed 5 % in the elastic stage. The precast–monolithic frame system based on a permanent shell formwork column is therefore equal in strength and stiffness to monolithic technology and eliminates the joint-zone deficiencies identified in the existing systems.

Keywords:

precast–monolithic concrete, permanent shell formwork, shell–core composite column, joint zone, comparative analysis, experimental substantiation

1. Introduction

More than 20–25 million m² of residential floor area is erected annually in the Republic of Uzbekistan, and 60–70 % of it falls on multi-storey buildings. Most of these buildings are built of monolithic reinforced concrete in removable formwork. The high cost of formwork operations, the extended construction period, the difficulty of controlling concrete quality on site and the dependence on weather conditions are the principal limitations of this technology. Precast–monolithic technology is regarded as a promising way of overcoming them: the frame elements are manufactured under factory conditions and are then united on site with cast-in-place concrete into a single spatial structure [1, 2].

Dozens of systems have been developed and certified worldwide in this direction. However, since each of them was formed around a particular structural idea, the set of systems covers not a continuous field but a number of areas that do not fully overlap. The systematic analysis carried out by Zoteyeva [3] revealed that the deficiencies of the most widespread solutions are concentrated precisely in the joint zone. Nevertheless, the literature review shows that no study has yet compared the structural limitations of the existing systems within a single set of criteria and linked the gap arising from that comparison directly with experimental verification.

The scientific basis of the present work was laid by the author's earlier studies: the influence of stage-by-stage erection and loading on the stress–strain state of multi-storey frame elements [20], the tests of columns made in permanent

formwork [18] and the tests of precast hollow-core slab models [19]. The calculation apparatus relies on the nonlinear deformation model developed for flexural reinforced concrete members [21].

The objective of this paper is to compare the precast–monolithic frame systems used in world practice against a single set of criteria, to substantiate the research gap they leave uncovered, to formulate a structural solution filling that gap and to verify its performance experimentally. The novelty lies in the direct coupling of the two stages: every limitation identified at the first stage is answered by a measured indicator obtained at the second.

2. Research methodology

2.1. Method of the analytical part

The analysis covered the precast–monolithic frame systems that have been applied on an industrial scale in world and CIS practice. The sources were the official documents and series albums of the systems, the technical manuals of the manufacturers [4], the normative documents [5–8, 23] and the generalising studies [3, 9]. Each system was described against five criteria: (1) the column solution; (2) the beam solution; (3) the floor solution; (4) the detailing of the joint zone; (5) the principal limitation reported in the sources. The systems were then grouped by these criteria and a matrix of development directions was compiled; the empty cell of the matrix was interpreted as the research gap.

2.2. Method of the experimental part

 <https://orcid.org/0009-0009-8061-3336>



The solution filling the identified gap was verified on three types of frame model. The models were manufactured at 1:4 scale on the basis of similarity theory [10, 11]. Since the materials of the model and of the prototype were identical, the scale factors for stress and modulus of elasticity were taken as $k\sigma = kE = 1$; with a geometric scale $kl = 4$ this gives $kP = kl^2 = 16$ and $kf = kl = 4$. Each model was two-bay and two-storey with an axial grid of 1300×1200 mm. Model M-1 is the conventional monolithic reference; model M-2 has columns formed in permanent reinforced concrete shell formwork, U-shaped precast-monolithic beams and a monolithic floor; model M-3 has the same columns and beams as M-2 but a floor made of precast hollow-core slabs.

The concrete was of class B25. The auxiliary specimens were tested in accordance with GOST 10180-2012 [12] and GOST 24452-80 [13]; the following characteristics were obtained: $R_{bm} = 30.84$ MPa, $R_{bn} = 29.60$ MPa, $R_b = 22.77$ MPa, $R_{bt,n} = 1.85$ MPa and $E_b = 2.9 \cdot 10^4$ MPa. The longitudinal reinforcement was of class A400 ($\sigma_y = 690$ N/mm², $\sigma_u = 750$ N/mm², $\delta = 20$ %). All data were obtained in an accredited laboratory.

Loading was applied in stages by DG-25 hydraulic jacks in accordance with GOST 8829-94 [14] and the test procedure of Zalesov and Klimov [15]; the step of each stage was taken equal to 0.1 of the design ultimate load. Strains were measured by ICh-100 dial gauges with an accuracy of 0.01 mm and cracks were measured with an MPB-2 optical microscope (0.02 mm). Four characteristic states were recorded for each model: the formation of the first crack (P_{cr}), the attainment of the normative crack width ($a_{cr} = 0.3$ mm — P_{acr}), the attainment of the limiting deflection ($f_{max} = L/200$ — P_f) and the loss of load-bearing capacity (P_u).

3. Results and Discussion

3.1. Comparative analysis of the precast-monolithic frame systems used in world practice

The results of the comparison against the five adopted criteria are given in Table 1. The systems are grouped not geographically but by their structural feature, which makes their limitations directly comparable with one another.

Table 1

Comparison of the precast-monolithic frame systems used in world practice against five criteria

System (country)	Column solution	Beam solution	Floor solution	Joint zone	Principal limitation identified
KUB-2.5 (Russia)	Precast multi-storey column	Beamless	Precast slab	“Slab-column” joint, monolithic ring	Frame performance depends on a single joint; no reserve of reliability
ARKOS (Belarus)	Precast column	Concealed beam inside the slab	Hollow-core slab	Keyed connection at the support	Support keys are executed without reinforcement
RECON (Russia)	Precast column	Beam with U-shaped loop bars	Hollow-core slab	Loop bars + monolithic key	Insufficient anchorage of the loop bars and insufficient key length (300 mm)
UDS (Russia)	Precast column	Precast-monolithic beam	Precast slab	Support joint	Support joint is not strengthened by additional reinforcement
Deltabeam (Finland)	Column with storey-wise slots	Steel-concrete composite Δ -beam	Hollow-core or monolithic slab	Welded steel connection	High steel consumption and amount of welding; dependence on a special factory product
IMS (former Yugoslavia)	Precast column	Post-tensioned beam	Precast slab	By post-tensioning	Special prestressing equipment and high qualification required
Structurapid (Italy)	Single-storey tubular column	T-shaped beam	Precast slab	Insertion into the column groove	Limited storey-to-storey connection of the tubular column
Thomas (USA)	1–3 storey column	Prestressed box-section beam	T-shaped slab + 76 mm topping	Lower corbel with steel collars	Large overall floor depth (0.95–0.99 m)
RPS (Bulgaria)	Precast column	Beamless	Two types of hollow-core slab	Through the groove at the slab end	Limited in-plane stiffness of the floor diaphragm
BGK (Ukraine)	Precast column	Beamless	Hexagonal slab	Mushroom-shaped capital	Punching of the slab in the capital zone governs the design
Filigran / Elementdecken-platten (Germany)	Wall system (no columns)	—	50–70 mm shell panels + monolithic core	Contact joint between shell and core	Wall rather than frame layout; free planning is restricted



Table 1 shows that in most systems the limitation is concentrated precisely in the joint zone, that is, at the place where the precast and the cast-in-place parts meet. The column, on the contrary, is treated in almost every system as a wholly factory-made product, and the composite action of precast and cast-in-place concrete within its cross-section is not used as a structural resource.

3.2. Development directions and the identified research gap

Generalisation of the data of Table 1 made it possible to distinguish four independent development directions of precast–monolithic technology: (1) beamless systems (KUB-2.5, RPS, BGK); (2) systems with concealed beams (ARKOS); (3) steel–concrete composite systems (Deltabeam); (4) wall and volumetric-block systems (Filigran, KOPE). The coverage of these directions with respect to the column, the beam and the floor is presented as a matrix in Table 2.

Table 2

Coverage of the development directions of precast–monolithic technology and the identified gap

Direction	Composite action of precast and cast-in-place concrete in the column	Composite action in the beam	Composite action in the floor	Formwork acting as part of the structure
1. Beamless (KUB-2.5, RPS, BGK)	—	no beam	+	—
2. With concealed beam (ARKOS)	—	+	+	—
3. Steel–concrete composite (Deltabeam)	—	+ (steel shell)	+	+ (in the beam only)
4. Wall / volumetric block (Filigran, KOPE)	no column	—	+	+ (in the wall only)
Proposed solution	+ (RC shell + monolithic core)	+ (U-shaped precast–monolithic beam)	+ (monolithic or precast–monolithic)	+ (in column, beam and floor)



Fig. 1. The proposed precast–monolithic frame structural solution and its joint zone

Table 2 shows the coverage and, at the same time, the gap: in none of the existing directions is the principle of load-bearing formwork extended as far as the column cross-section. In the Deltabeam system this principle is realised only in the beam (a steel shell), and in the Filigran system only in the wall. In other words, the concept of a column formed inside thin-walled permanent reinforced concrete shell formwork, acting as a single composite section together with its inner monolithic core and working jointly with a U-shaped precast–monolithic beam and a precast floor, has not been developed. This gap defines the subject of the present study.

3.3. The proposed structural solution

In order to fill the identified gap, a solution consisting of the following three elements was formulated. The column is a thin-walled reinforced concrete shell formwork, reinforced with basalt bars, whose wall thickness is about one eighth of the cross-sectional dimension and whose interior is filled with cast-in-place concrete on site. Having performed the function of formwork, the shell becomes a working part of the cross-section and confines the core concrete laterally. The beam is a precast–monolithic member whose lower part is manufactured

in the factory with a U-shaped section and whose upper part is cast together with the floor; the U-shaped shell remains open for placing the reinforcement, so that the contact joint lies within the effective depth of the section and not in the tensile zone. The floor is either a monolithic slab (variant M-2) or a topping cast over precast hollow-core slabs (variant M-3).

3.4. Experimental verification of the proposed solution

In all three models the failure developed in accordance with the theory of limit states: cracks first appeared in the tensile zone of the beams and in the joint zones, then the tensile reinforcement reached its yield limit, and finally the concrete of the compressed zone was crushed [16, 17]. In models M-2 and M-3 the columns behaved as two-layer composite members: no signs of slip were observed at the interface between the shell and the core, and in the elastic stage (load $\leq 0.5 \cdot P_u$) the difference between their longitudinal strains did not exceed 5 %. This confirms that the shell and the core work as a single composite section in a regime of equal strains.

The characteristic loads determined for the three models and their ratios are given in Table 3.



Table 3

Results of testing the frame models under static loads

Indicator	M-1 (reference, monolithic)	M-2 (shell-formwork column, monolithic floor)	M-3 (shell-formwork column, precast floor)
Cracking load P_{arc} , kN	43.62	36.38	43.62
Load at $a_{arc} = 0.3$ mm, P_{arc} , kN	94.30	94.30	94.30
Load at limiting deflection P_f , kN	122.15	125.84	104.42
Ultimate load P_u , kN	130.50	134.12	119.64
P_{arc} / P_u	0.723	0.703	0.788
P_u relative to M-1, %	100	102.8	91.7
P_f relative to M-1, %	100	103.0	85.5
P_{rc} relative to M-1, %	100	83.4	100
Deflection at $P = 101.54$ kN relative to M-1, %	100	108...117	149...189



Fig. 2. “Load–deflection” relationships for the three models and comparison of the ultimate loads

When the model parameters were converted to the prototype building by means of the adopted scale factors, the column section became 400×400 mm, the shell wall thickness 50 mm, the core section 300×300 mm, the hollow-core slab thickness 220 mm (void diameter 160 mm) and the longitudinal reinforcement 4Ø32 A400 — all of which correspond exactly to the dimensions of standard products in current use. The ratio of the limiting deflections was 24.0/6.0

= 4, that is, exactly equal to the adopted factor k_f . This independently confirms that the similarity conditions were satisfied.

Discussion

The principal result of the study is the link between the analytical and the experimental stage: every limitation identified in Table 1 can be answered by a measured indicator from Table 3. This correspondence is summarised in Table 4.

Table 4

Limitations identified in the existing systems and the response of the proposed solution

Identified limitation (system)	Structural response in the proposed solution	Experimental confirmation
Unreinforced key at the support joint (ARKOS)	The joint zone is reinforced inside the U-shaped shell of the beam; the key is crossed by reinforcement	Horizontal cracks appeared in the joint zone only at the final loading stages; P_{arc} was identical in all three models (94.30 kN)
Insufficient anchorage of the loop bars (RECON)	The beam reinforcement is carried continuously inside the U-shaped shell; loop bars are not required	Failure was governed by yielding of the reinforcement, not by anchorage
Frame performance depends on a single joint, no reserve (KUB-2.5)	The column–beam–floor triad forms three consecutive independent connections	Failure developed gradually, in a ductile manner and with warning cracks
High steel consumption and amount of welding (Deltabeam)	The shell is made of thin-walled reinforced concrete rather than steel; basalt reinforcement is used; welding is not required	$P_u = 134.12$ kN — 2.8 % higher than the reference, without a steel shell
Composite action of precast and cast-in-place concrete in the column section is not used (all systems)	The shell becomes a working part of the section and confines the core laterally	The difference between the longitudinal strains of shell and core is ≤ 5 %; no slip was observed



The crack resistance of model M-2 requires a separate comment: its first crack was recorded at a load 16.6 % lower than that of the reference (36.38 kN). This is explained by the fact that in the M-2 beam the horizontal contact joint between the precast and the cast-in-place layers lies in the upper part of the tensile zone and that the first crack was localised precisely along this joint [22]. It is essential, however, that this did not adversely affect the ultimate load-bearing capacity: the load corresponding to the normative crack width was the same in all three models (94.30 kN), which means that under normal service conditions all three solutions are equally reliable.

Variant M-3 gave a result 8.3 % lower in terms of ultimate load and 14.5 % lower in terms of the load at limiting deflection, and its deflections were 1.5–1.9 times greater than those of the reference. The reason is the more hinge-like connection of the precast hollow-core slabs to the beams and the lower stiffness of the floor diaphragm. This independently confirms the conclusion recorded in Table 1 for beamless systems and systems with precast floors, namely the limited in-plane stiffness of the diaphragm. Accordingly, M-2 was adopted as the principal recommended solution and M-3 as an alternative one.

The results obtained do not contradict the principles of shear resistance along the contact joint set out in Section 10 of EN 1992-1-1 [5], in the provisions of ACI 318-19 concerning precast structures [6, 23] and in fib Model Code 2010 [7]: the absence of slip at the shell–core interface indicates that the roughened-surface condition required by these documents was satisfied. At the same time, the current national normative documents [8] contain no direct provisions for assessing the crack resistance and the stiffness of such composite sections; the present results may serve as an initial basis for filling this gap.

Limitations of the study. The investigation was restricted to static loads; behaviour under seismic (cyclic) actions requires a separate study. In addition, at a scale of 1:4 the bond law between the reinforcement and the concrete does not obey complete similarity, and therefore the results concerning crack spacing and crack width should be extrapolated by means of correction factors.

4. Conclusion

1. The precast–monolithic frame systems used in world and CIS practice were compared against five criteria, and it was established that their limitations are concentrated mainly in the joint zone, that is, at the place where the precast and the cast-in-place parts meet, whereas the column section is treated in almost every system as a wholly factory-made product.

2. Four development directions of precast–monolithic technology were distinguished, and it was shown that in none of them is the principle of load-bearing formwork extended as far as the column section. To fill this gap, a solution comprising a column in thin-walled permanent reinforced concrete shell formwork, a U-shaped precast–monolithic beam and a precast–monolithic floor was proposed.

3. Variant M-2 of the proposed solution gave a result 2.8 % higher than the monolithic reference in terms of ultimate load (134.12 kN against 130.50 kN) and 3.0 % higher in terms of the load at limiting deflection, while its deflections

differed from those of the reference by only 8–17 %.

4. In the composite column with permanent shell formwork the shell and the monolithic core act jointly and without slip in the elastic stage — the difference between their longitudinal strains did not exceed 5 %. This experimentally justifies taking the shell into account as a working part of the cross-section.

5. The load corresponding to the normative crack width was 94.30 kN in all three models. Hence, under normal service conditions all three structural solutions are equally reliable.

6. When converted to the prototype building, the model parameters coincided exactly with the dimensions of standard products in current use, and the ratio of the limiting deflections was equal to $k_f = 4$. This confirms that the similarity conditions were satisfied and makes it possible to transfer the results to real buildings.

References

- [1] Golyshev A.B., Polishchuk V.P., Sungatullin Ya.G., Bachinsky V.Ya. et al. Design and manufacture of precast–monolithic structures / ed. by A.B. Golyshev. – Kyiv: Budivelnik, 1982. – 152 p.
- [2] Mordich A.I., Belevich V.N., Simbirkin V.N., Navoy D.I. Experience of practical application and results of full-scale tests of the BelNII precast–monolithic frame // Bulletin of Construction Engineering. – 2004. – No. 8. – pp. 8–12.
- [3] Zoteyeva E.E. Complex assessment and improvement of structural and technological solutions of civil monolithic and precast–monolithic buildings: Cand. Tech. Sci. diss.: 05.23.08. – Yekaterinburg: Ural Federal University, 2018. – 148 p.
- [4] Peikko Group. DELTABEAM® Composite Beam. Technical Manual. – Lahti: Peikko Group Corporation, 2023.
- [5] EN 1992-1-1:2004. Eurocode 2: Design of concrete structures. Part 1-1: General rules and rules for buildings. – Brussels: CEN, 2004. – 226 p.
- [6] ACI 318-19. Building Code Requirements for Structural Concrete and Commentary. – Farmington Hills, MI: American Concrete Institute, 2019. – 624 p.
- [7] fib Model Code for Concrete Structures 2010. – Berlin: Ernst & Sohn, 2013. – 434 p.
- [8] ShNK 2.03.01-24. Concrete and reinforced concrete structures. – Tashkent: Ministry of Construction of the Republic of Uzbekistan, 2024.
- [9] Jasienko J., Oleinik A., Pisznyak I. Theory of strengthening and restoration of precast–monolithic buildings using permanent formwork // Archives of Civil and Mechanical Engineering. – 2010. – Vol. 10.
- [10] Buckingham E. On physically similar systems; illustrations of the use of dimensional equations // Physical Review. – 1914. – Vol. 4, No. 4. – pp. 345–376.
- [11] Sabnis G.M., Harris H.G., White R.N., Mirza M.S. Structural modeling and experimental techniques. – Englewood Cliffs, NJ: Prentice-Hall, 1983. – 585 p.
- [12] GOST 10180-2012. Concretes. Methods for strength determination using reference specimens. – Moscow: Standartinform, 2013. – 30 p.
- [13] GOST 24452-80. Concretes. Methods of prismatic compressive strength, modulus of elasticity and Poisson's



ratio determination. – Moscow: Publishing House of Standards, 1980. – 16 p.

[14] GOST 8829-94. Reinforced concrete and concrete factory-made building products. Loading test methods. – Moscow: Standartinform, 1997. – 26 p.

[15] Zalesov A.S., Klimov Yu.A. Strength of reinforced concrete structures under the action of transverse forces. – Kyiv: Budivelnik, 1989. – 104 p.

[16] Gvozdev A.A. Calculation of the load-bearing capacity of structures by the limit equilibrium method. – Moscow: Gosstroyizdat, 1949. – 280 p.

[17] Murashev V.I. Crack resistance, stiffness and strength of reinforced concrete. – Moscow: Mashstroyizdat, 1950. – 268 p.

[18] Khamrokulov U.D., Madatov I.A., Kamoliddinov D. Experimental studies on testing columns made in permanent formwork // Problems of Architecture and Construction. – 2025. – No. 2. – pp. 75–77.

[19] Khamrokulov U.D. Determination of the load-bearing capacity of a precast hollow-core slab model // Problems of Architecture and Construction. – 2025. – No. 3. – pp. 58–60.

[20] Khamrokulov U.D. The influence of stage-by-stage erection and loading on the stress–strain state of multi-

storey frame building elements: PhD diss. – Samarkand: SamSUACE, 2022. – 142 p.

[21] Usmanov V.F. Strength, crack resistance and deformation of flexural bar elements of reinforced concrete structures: Doct. Tech. Sci. (DSc) diss. – Samarkand: SamSUACE, 2023. – 268 p.

[22] Smolyago G.A. Deformability and crack resistance of flexural reinforced concrete elements: monograph. – Belgorod: BSTU named after V.G. Shukhov, 2009. – 224 p.

[23] PCI Design Handbook: Precast and Prestressed Concrete. 8th ed. – Chicago: Precast/Prestressed Concrete Institute, 2017.

Information about the author

**Ulugbek
Khamrokulov**

Samarkand State Technical
University named after Mirzo
Ulugbek, Department of Civil
engineering, DSc student, PhD
E-mail: midimin@mail.ru
Tel.: +998 91 552-96-64
<https://orcid.org/0009-0009-8061-3336>



O. Khamidov, Sh. Samatov, B. Gayratov, A. Djuraev

Improving the reliability of the TD802 direct-current traction generator of the ChME3 shunting diesel locomotive through insulation replacement.....118

U. Turdialiev, D. Boykishieva

Assessing the impact of integrating lean manufacturing methods into the quality management system on the efficiency of a mechanical engineering enterprise.....125

B. Sobirov, N. Botirova

Use of mineral fillers for PVC-based linoleum production and investigation of their physical and mechanical properties.....130

G. Uzakov, M. Majidova

Development of an artificial intelligence-based adaptive digital twin method for early fault detection and energy loss reduction in solar photovoltaic systems.....133

U. Khamrokulov

Precast-monolithic frame building system based on permanent reinforced concrete shell formwork: Analysis of world practice and experimental substantiation.....138

Z. Kurbonov, S. Bozorboev

Mechanically induced surface modification of diabase aggregate for high-performance concrete.....144