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JUSTIFICATION OF THE INTEGRATED SUPPORT SCHEME FOR THE PORTAL OF AN INCLINED RAMP UNDER A COMBINED OPEN-PIT MINING SYSTEM

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The article addresses the issues of improving the stability of the portal and the initial section of an inclined ramp constructed during the transition of an open pit to a combined mining system. It is in this zone that the effects of rock mass unloading, roof and sidewall instability, water inflow, as well as additional dynamic impacts caused by drilling-and-blasting operations and the movement of mine transport occur simultaneously. The aim of the study was to substantiate an integrated support scheme that ensures the early formation of a load-bearing rock mass contour, reduces deformations in the near-contour zone, and preserves technological compatibility with both drilling-and-blasting and mechanized excavation methods. The study was carried out on the basis of field observations, laboratory testing of concrete and injection materials, instrumental monitoring of deformations, and numerical modeling in the FLAC3D software environment. The proposed scheme includes the sequential application of a primary shotcrete layer, installation of advance inclined rock bolts along the excavation direction, arrangement of a staggered grid of injection anchors, concreting of a section at least 20 m long, and primary and control grouting after the concrete reaches 75% of its design strength. It was established that the application of this scheme ensures a 2.3-fold reduction in water inflow, a 1.8-fold decrease in roof deformations compared with the control section, and a reduction in the maximum calculated displacements from 42 to 12 mm when rock bolting and injection reinforcement are used jointly. The obtained results confirm the feasibility of applying a combined support system in unstable host rocks and make it possible to recommend it for practical implementation in the construction of inclined ramp portals.

Keywords: inclined ramp, portal, combined mining system, advance rock bolting, shotcrete, injection anchors, excavation stability, numerical modeling.

КАРЬЕРДІ ҚҰРАМДАСТЫРЫЛҒАН ИГЕРУ ЖҮЙЕСІ ЖАҒДАЙЫНДА КӨЛБЕУ СЪЕЗД ПОРТАЛЫН БЕКІТУДІҢ КЕШЕНДІ СҰЛБАСЫН НЕГІЗДЕУ

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Мақалада карьердің құрамдастырылған игеру жүйесіне көшуі кезінде салынатын көлбеу съездің порталы мен бастапқы учаскесінің орнықтылығын арттыру мәселелері қарастырылған. Дәл осы аймақта массивтің босауының әсері, төбе мен борттардың орнықсыздығы, сулану, сондай-ақ бұрғылап-жару жұмыстары мен технологиялық көліктің қозғалысына байланысты қосымша динамикалық әсерлер бір мезгілде байқалады. Зерттеудің мақсаты массивтің көтергіш контурын ерте қалыптастыруды, контур маңы аймағындағы деформацияларды азайтуды және бұрғылап-жару, сондай-ақ механикаландырылған өту жұмыстарымен технологиялық үйлесімділікті сақтауды қамтамасыз ететін кешенді бекіту сұлбасын негіздеу болды. Зерттеу далалық бақылаулар, бетон мен инъекциялық құрамдарды зертханалық сынау, деформацияларды аспаптық бақылау және FLAC3D бағдарламалық ортасында сандық модельдеу негізінде орындалды. Ұсынылған сұлбаға алғашқы набрызг-бетон қабатын кезең-кезеңімен жағу, қазба бойымен алға оздырылған көлбеу анкерлерді орнату, инъекциялық анкерлердің шахматтық торын монтаждау, ұзындығы кемінде 20 м учаскені бетондау және бетон жобалық беріктігінің 75 %-ына жеткеннен кейін бастапқы және бақылау нагнетаниесін жүргізу кіреді. Аталған сұлбаны қолдану су ағынын 2,3 есе азайтуды, бақылау учаскесімен салыстырғанда төбе деформацияларын 1,8 есе төмендетуді және анкерлеу мен инъекциялық бекітуді бірлесіп қолданған кезде есептік ең үлкен орын ауыстыруларды 42-ден 12 мм-ге дейін қысқартуды қамтамасыз

ететіні анықталды. Алынған нәтижелер орнықсыз жанасушы жыныстарда бекітудің құрамдастырылған жүйесін қолданудың орындылығын растайды және оны көлбеу съезд порталдарын салу кезінде тәжірибеде енгізуге ұсынуға мүмкіндік береді.

Түйін сөздер: көлбеу съезд, портал, құрамдастырылған игеру жүйесі, оздырылған анкерлеу, набрызг-бетон, инъекциялық анкерлер, қазбалардың орнықтылығы, сандық модельдеу.

ОБОСНОВАНИЕ КОМПЛЕКСНОЙ СХЕМЫ КРЕПЛЕНИЯ ПОРТАЛА НАКЛОННОГО СЪЕЗДА ПРИ КОМБИНИРОВАННОЙ СИСТЕМЕ РАЗРАБОТКИ КАРЬЕРА

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В статье рассмотрены вопросы повышения устойчивости портала и начального участка наклонного съезда, сооружаемого при переходе карьера к комбинированной системе разработки. Именно в этой зоне одновременно проявляются влияние разгрузки массива, неустойчивость кровли и бортов, обводнение, а также дополнительные динамические воздействия, обусловленные буровзрывными работами и движением технологического транспорта. Цель исследования заключалась в обосновании комплексной схемы крепления, обеспечивающей раннее формирование несущего контура массива, снижение деформаций приконтурной зоны и сохранение технологической совместимости как с буровзрывной, так и с механизированной проходкой. Исследование выполнено на основе полевых наблюдений, лабораторных испытаний бетона и инъекционных составов, инструментального контроля деформаций и численного моделирования в программной среде FLAC3D. Предлагаемая схема включает последовательное нанесение первичного слоя набрызг-бетона, установку опережающих наклонных анкеров по ходу выработки, монтаж шахматной сетки инъекционных анкеров, бетонирование участка длиной не менее 20 м и проведение первичного и контрольного нагнетания после достижения бетоном 75 % проектной прочности. Установлено, что применение данной схемы обеспечивает снижение водопритока в 2,3 раза, уменьшение деформаций кровли в 1,8 раза по сравнению с контрольным участком и сокращение максимальных расчетных смещений с 42 до 12 мм при совместном использовании анкеровки и инъекционного укрепления. Полученные результаты подтверждают целесообразность применения комбинированной системы крепления в неустойчивых вмещающих породах и позволяют рекомендовать ее для практического внедрения при сооружении порталов наклонных съездов.

Ключевые слова: наклонный съезд, портал, комбинированная система разработки, опережающая анкеровка, набрызг-бетон, инъекционные анкеры, устойчивость выработок, численное моделирование.

Introduction. The transition from open-pit mining to a combined deposit development system requires particular attention to the condition of the inclined ramp portal. In this zone, the most unfavorable factors are concentrated over a short distance: changes in the stress state of the rock mass due to excavation, the presence of fractured and weakened zones, the impact of atmospheric and groundwater, as well as variable loads from excavation equipment and transport vehicles. Under insufficient support stiffness, the portal becomes a high-risk area where roof collapses, sidewall failures, and floor destruction are most likely [1].

Analysis of existing solutions shows that traditional massive lining systems (segmental or tubbing linings) provide high reliability; however, they are characterized by high cost, labor-intensive installation, and limited adaptability to the rectangular cross-section of ramps [2]. Systems based on early shotcrete application and rock bolting are more flexible and widely used in tunneling and underground excavation practice, [3, 4], but under conditions of high fracturing and water inflow, they do not always prevent local failures without advance face reinforcement.

Modern concepts emphasize the importance of early stabilization of the near-contour zone and the formation of a load-bearing rock-support system [5, 6]. For excavations in unstable rocks, the effectiveness of combined support systems—integrating rock bolting, shotcrete, and injection strengthening—has been substantiated [7, 8]. However, the issue of technological coordination of all support elements specifically for inclined ramp portals in the transition zone from open-pit to underground mining remains insufficiently studied [9, 10].

The research object is the portal and the initial section of a rectangular inclined ramp constructed in

alternating sandstones, siltstones, and argillites with varying degrees of weathering, fracturing, and water saturation. The subject of the study is the stability evolution of the near-contour zone under sequential application of shotcrete, advance rock bolting, injection reinforcement, and monolithic concreting.

The aim of the study is to substantiate a comprehensive support scheme for the inclined ramp portal under a combined mining system. The hypothesis is that a staged combination of inclined advance bolts, a staggered injection bolting system, and controlled concreting over a length of at least 20 m reduces deformation, improves rock mass stability, and ensures technological reliability of the portal.

The scientific novelty of the study lies not in the isolated use of shotcrete, rock bolts, grouting, or monolithic concrete, which are established support measures, but in their engineering integration into a staged support sequence specifically for the transition zone of an inclined ramp portal. The proposed sequence links advance inclined bolting, a staggered injection-bolt grid, and subsequent concreting to a measurable technological criterion—achievement of at least 75% of the design concrete strength before control grouting. In contrast to conventional solutions in which individual support elements are designed and installed largely independently, the proposed approach coordinates the time of support engagement with the development of near-contour deformation and water inflow while preserving the rectangular ramp geometry and compatibility with drill-and-blast excavation. The study therefore evaluates the combined effect of the support sequence rather than presenting any of its individual elements as a new support technology.

Materials and Methods. The study was conducted for the portal and the initial 20 m section of the inclined ramp. The geological environment consists of interbedded sandstones, siltstones, and argillites. The near-contour zone contains one to three fracture sets, often subparallel to bedding, as well as local softened and water-bearing zones [5, 6].

The methodology included field, laboratory, instrumental, and numerical investigations.

The experimental program was organized as a comparative engineering assessment of the control and reinforced portal sections. Field observations, laboratory strength testing, instrumental deformation monitoring, and numerical calculations were treated as mutually independent evidence streams. Because the available field dataset contains aggregated monitoring values rather than a complete set of raw replicate measurements for every observation point, the article does not infer sample sizes, standard deviations, or confidence intervals that cannot be reconstructed from the source records. Statistical interpretation is therefore limited to the reported comparative indicators, model–field deviation, coefficient of determination, and sensitivity analysis; this limitation is explicitly considered when interpreting the results.

Field observations recorded fracturing intensity, water inflow, and deformation behavior of the roof and sidewalls. Ground-penetrating radar (GPR) “OKO-3” (400 MHz antenna) was used to identify weakened and water-saturated zones. Deformations were monitored using strain gauges and inclinometers with an accuracy of $\pm 10^{-5}$. Seismoacoustic monitoring (RIPAS system) was applied to assess stress redistribution.

Laboratory tests determined the strength characteristics of shotcrete and injection materials. Specimens (150×150×150 mm) were tested according to GOST 10180-2012 using a hydraulic press (capacity 2000 kN). Strength gain was described by the exponential relationship:

$$\sigma(t) = \sigma 28\left(1 - e^{-kt}\right)$$

A critical threshold of 75% design strength was adopted as the criterion for safe transition to control grouting. [3, 4].

The engineering support scheme consisted of several sequential operations. Immediately after excavation of the contour, a primary layer of shotcrete with a thickness of 5–7 cm was applied. Subsequently, advance inclined rock bolts with a length of 3.0–3.5 m were installed along the excavation at an angle of 10–15° to the ramp axis, with a spacing of 1.2–1.5 m. This was followed by the installation of a staggered grid of injection bolts with a spacing of 1.5 × 1.5 m and an injection pressure of 0.3–0.5 MPa. The next stage involved concreting of the roof, sidewalls, and floor over a section with a length of at least 20 m. After the concrete reached 75% of its design strength, primary and control grouting were performed to further consolidate the near-contour zone and reduce the filtration permeability of the rock mass [9, 10].

For validation of the proposed scheme, numerical modeling was carried out using the FLAC3D 7.0 software package. The computational model represented a 30 m long section with a cross-section of 5 × 5 m. The mesh contained approximately 70,000 finite elements with refinement in the portal area. The base was assumed to be rigidly fixed, elastic boundary conditions were applied at the lateral boundaries, and the external load corresponded to the weight of the overlying rock mass. The following parameters were

adopted: for sandstones, uniaxial compressive strength σc = 55 MPa and elastic modulus E = 25 GPa; for siltstones, σc = 28 MPa and E = 12 GPa; for argillites, σc = 18 MPa and E = 8 GPa. Three scenarios were considered: without reinforcement, with advance rock bolting, and with combined application of bolting and injection. Additionally, the Q-system was used for engineering characterization of rock-mass quality. Model verification was performed by comparing calculated roof displacements with instrumental observations for the same support scenarios. Calibration was carried out by checking whether the adopted deformation parameters reproduced the measured displacement range without changing the geological structure of the model. Model adequacy was assessed using relative model–field deviation and the coefficient of determination (R^2), while parameter uncertainty was examined through a sensitivity analysis in which the rock deformation modulus was varied by ±20%. Inferential significance levels were not reported because the archived monitoring dataset does not provide a uniform number of independent raw replicates for all compared points.

Results and Discussion. Field and laboratory studies showed that the primary shotcrete layer achieved, on average, 72% of the design strength after 3 days and exceeded 75% after 5 days, which is sufficient to proceed to control grouting and further development of the support scheme. Monolithic concrete reached a similar threshold after 9 days. This curing regime proved to be critically important, as it allowed maintaining the construction schedule while preventing injection effects on an insufficiently hardened support contour [3, 4]. The strength development of shotcrete and monolithic concrete is presented in Figure 1.

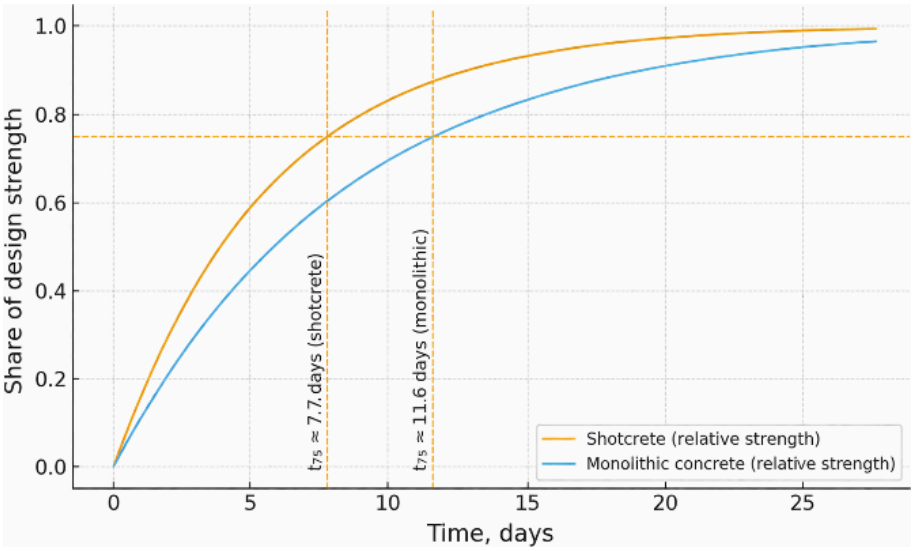


Fig. 1 - Strength development and the allowable grouting point at 75% of design strength

Instrumental observations in the portal zone showed that the use of advance inclined rock bolts leads to a significant reduction in roof deformations already at the early stage of excavation. Compared to the control section, displacements decreased by a factor of 1.8. After incorporating a staggered grid of injection bolts into the support scheme, not only a further reduction in deformations was observed, but also a 2.3-fold decrease in water inflow, indicating partial closure of fractures and a reduction in the filtration permeability of the near-contour zone [10].

The results of numerical modeling confirmed the trends identified under field conditions. In the absence of reinforcement, maximum roof displacements reached 42 mm, indicating a highly unfavorable state of the rock mass for the portal section. The scenario with only advance rock bolting reduced this value to 25 mm, whereas the combined application of bolting and injection strengthening decreased it to 12 mm. Thus, the calculated reduction in deformation relative to the unsupported state was approximately 3.5 times. The stress distribution pattern in the model indicated load redistribution and the formation of a more stable load-bearing framework around the excavation contour. The zone of plastic deformation was reduced, and stress concentrations shifted into the reinforced region, where they were absorbed by the bolting system and the strengthened rock mass [5, 6, 10].

Comparative scenarios of roof deformation based on modeling results are presented in Figure 2.

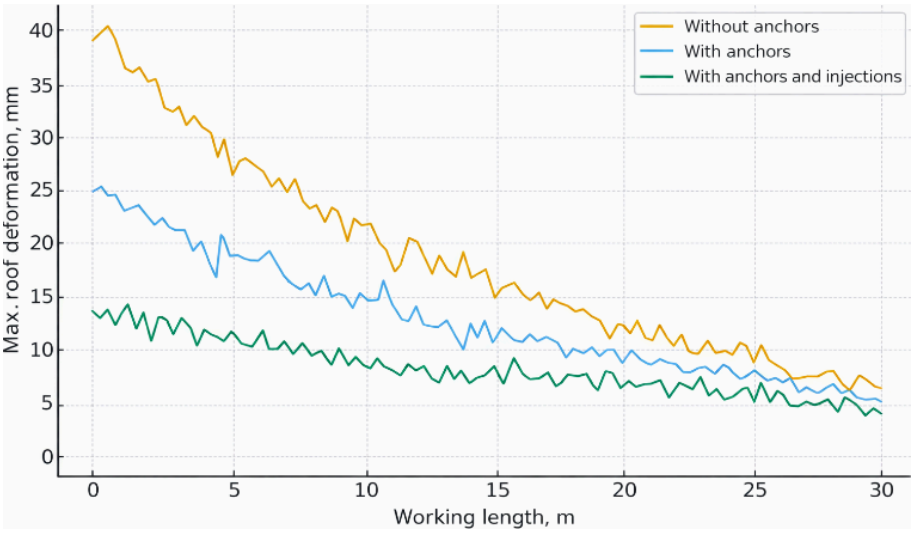


Fig. 2 - Roof deformation scenarios based on FLAC3D modeling results

Comparison of numerical modeling results with field observations showed that the calculated deformation trend reproduced the measured response for the considered support scenarios. The relative discrepancy between calculated and measured roof displacements did not exceed 15%, and the coefficient of determination for the calculation–measurement relationship was $R^2 \geq 0.90$. These indicators are used here as engineering validation metrics rather than as a substitute for inferential statistics. The agreement is considered sufficient for comparative assessment of the three support scenarios within the investigated portal section, while extrapolation beyond the studied geological conditions should be made with caution.

The uncertainty analysis focused on the deformation modulus because this parameter strongly controls calculated convergence and is sensitive to rock-mass heterogeneity. A ±20% variation in the adopted deformation modulus changed calculated displacements by approximately 12–15%, while the ranking of the three support scenarios and the overall stress–deformation pattern remained unchanged. Thus, the principal engineering conclusion—the lower deformation obtained for the combined bolting-and-injection scenario—was robust within the tested parameter interval. The analysis does not eliminate uncertainty associated with local fractures, moisture variation, or construction quality; these factors remain a limitation of the model and require field monitoring during implementation.

The comparison of calculated and field data is presented in Figure 3.

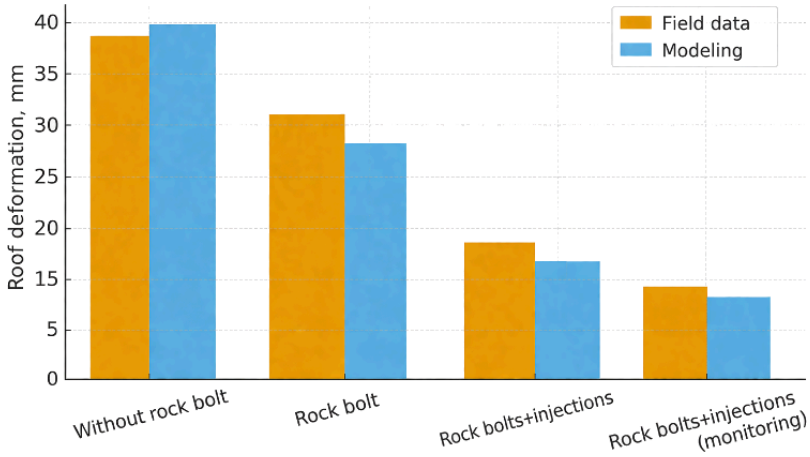


Fig. 3 - Comparison of modeling results and field observations

The obtained results are consistent with current concepts of active and combined support, according to which early stabilization of the near-contour rock mass and coordinated interaction of rock bolts and shotcrete can reduce the extent of yielding and deformation. The distinction of the present study is the adaptation of this principle to an inclined-ramp portal at the open-pit-to-underground transition and the explicit sequencing of

support operations under fractured and water-bearing conditions. The proposed scheme does not claim a new type of bolt, shotcrete, or grouting material. Its contribution is the experimentally and numerically evaluated technological sequence: primary shotcrete → advance inclined bolts → staggered injection bolts → monolithic concreting → control grouting after the 75% strength criterion is reached. This sequence reduced the calculated maximum roof displacement from 42 mm in the unsupported model to 25 mm with advance bolting and to 12 mm with combined bolting and injection, while field observations showed a 2.3-fold reduction in water inflow. These quantitative indicators define the engineering effect of integration and distinguish the proposed procedure from the isolated application of conventional support elements.

In practical terms, this means that within the first 10–15 m from the portal, where maximum displacements are observed, it is advisable to apply an increased density of rock bolting and ensure more rigorous control of injection filling. In zones of increased fracturing and water inflow, ground-penetrating radar diagnostics and exploratory drilling should be carried out prior to excavation [8, 10]. A limitation of the study is that the numerical model covered a 30 m section and used averaged rock parameters. In real conditions, local zones of fragmentation, variations in moisture content, and curing temperature regimes may affect the final strengthening effect; therefore, continuous instrumental monitoring is required during implementation of the technology.

The main limitations of the study are the single investigated portal, the 30 m numerical domain, the use of averaged mechanical properties for lithological units, and incomplete availability of raw replicate monitoring records. Accordingly, the results should be interpreted as a validated engineering case study rather than as a universal statistical law. Further verification should include additional portal sites, documented replicate measurements at fixed monitoring stations, reporting of sample size and dispersion for laboratory and field data, and calibration against three-dimensional fracture geometry and groundwater conditions.

Table 1 - Key quantitative results of the evaluation of the support scheme effectiveness

Parameter	Condition/Scenario	Value	Engineering Interpretation
Shotcrete strength development	After 3 days	72% of design strength	The contour gains early stiffness but still requires curing before control grouting
Shotcrete strength development	After 5 days	>75% of design strength	Transition to control grouting is permissible
Monolithic concrete strength development	After 9 days	75% of design strength	The section is ready for further technological operations
Water inflow	After installation of injection grid	Reduction by 2.3 times	Filtration permeability decreases and risk of rock softening is reduced
Maximum roof deformation	Without reinforcement	42 mm	The near-contour zone does not meet required stability conditions
Maximum roof deformation	Advance bolting only	25 mm	Bolting stabilizes the contour but does not eliminate the influence of water and fracturing
Maximum roof deformation	Bolting + injection reinforcement	12 mm	The most stable load-bearing contour is formed
Model–field agreement	Calculation vs. field data	Deviation ≤15%; $R^2 \geq 0.9$	The modeling approach is suitable for engineering evaluation of support options

The data presented in Table 1 confirm that the greatest effect is achieved not through the isolated application of individual support measures, but through their sequential combination within a unified technological cycle.

Conclusion. The conducted study demonstrated that the most complex and unstable section of an inclined ramp is its portal and initial segment, where stress relief, increased rock fracturing, water inflow, and dynamic loads occur simultaneously. Under such conditions, the use of a single type of support is generally insufficient

to ensure the required reliability and long-term stability of the excavation.

It has been established that a comprehensive support scheme, including primary shotcrete, advance rock bolting, injection strengthening, and subsequent concreting, makes it possible to form a more stable load-bearing contour around the excavation and reduce the risk of local deformation development. Achieving 75% of the design concrete strength can be considered a justified technological criterion for performing control grouting and transitioning to the next stage of rock mass stabilization.

The practical significance of the study lies in the fact that the proposed approach can be applied in the construction of inclined ramps in unstable rock masses without abandoning drill-and-blast technology, ensuring increased excavation stability, reduced probability of deformation-related failures, and an overall improvement in the safety of tunneling operations.

The scientific contribution is the substantiation of a staged support algorithm in which the timing and interaction of conventional support elements are coordinated with measured deformation, water inflow, and concrete strength development. Within the investigated portal, this integrated sequence produced a stable reduction in calculated deformation and acceptable agreement between numerical and field observations, providing a reproducible basis for selecting support stages under comparable fractured and water-bearing rock-mass conditions.

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