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RESEARCH ARTICLE

REDUCING THE STEEL CONSUMPTION OF A LONG-SPAN AUTOMOBILE CENTRE FRAME IN A SEISMIC REGION

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Abstract. *Steel frames account for a major share of the load-bearing cost of long-span commercial buildings in seismically active regions, and the sources of excess steel consumption in such projects remain poorly documented on the basis of as-built design data. The study quantifies feasible reductions of steel consumption for a two-storey automobile centre with a 17 m column-free showroom erected in Tashkent region, Uzbekistan, with seismic intensity 7. A plane-frame numerical model of the transverse frame was developed in Python, checked by 25 automatic verification tests, cross-validated with an independent finite element library with agreement within 0.01%, and verified against the design force envelopes, the deviations being 7-12% for the moments and shears of the governing members and within 13% for the column axial forces. Seismic action was evaluated by the response spectrum method of the national code KMK 2.01.03-19 for each block separated by antiseismic joints. The as-built frame consumes 224.4 t of steel, or 55.9 kg/m² of the total floor area. The design base shear equals 3.7-4.5% of the block weight, whereas the interstorey drift of the worst service-block frame reaches 1/189 against the code limit of 1/200, so stiffness governs the frame proportions. Five measures were assessed: section resizing saves 33.8 t, lattice girders of cold-formed hollow sections save 22.4 t, composite action of floor beams saves 16.5 t, a braced service block saves 8.4 t, and the higher steel grade S345 brings no saving. A consistent combination of the measures reduces consumption by 47.3 t, or 21.1%, to 44.1 kg/m². The results support stiffness-oriented design of long-span buildings in areas of moderate seismicity.*

Keywords: *steel frame; earthquake resistance; response spectrum method; interstorey drift; lattice girder; composite beam.*

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ҒЫЛЫМИ МАҚАЛА

СЕЙСМИКАЛЫҚ АУДАНДАҒЫ ҮЛКЕН АРАЛЫҚТЫ АВТООРТАЛЫҚТЫҢ БОЛАТ ҚАҢҚАСЫНЫҢ МЕТАЛЛ СЫЙЫМДЫЛЫҒЫН ТӨМЕНДЕТУ

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Аңдатпа. *Сейсмикалық аудандардағы үлкен аралықты коммерциялық ғимараттарда көтергіш конструкциялар құнының негізгі бөлігін болат қаңқалар құрайды, алайда мұндай нысандардың артық металл шығынының көздері жүзеге асырылған жобалардың деректері бойынша сирек құжатталады. Жұмыста Ташкент облысында MSK-64 шкаласы бойынша сейсмикалығы 7 балл алаңда салынған, аралығы 17 м бағанасыз шоурумы бар 2 қабатты автоорталық үшін болат шығынын азайту резервтеріне сандық баға берілді. Көлденең раманың жазық сандық моделі Python тілінде жүзеге асырылды, 25 автоматты верификациялық тестпен тексерілді, тәуелсіз ақырлы элементтік кітапханамен 0,01% шегіндегі сәйкестікпен салыстырылды және жобалық қамтушы эпюралар бойынша верификацияланды: ауытқулар негізгі элементтердің моменттері мен көлденең күштері бойынша 7-12%, бағаналардың бойлық күштері бойынша 13% шегінде. Сейсмикалық әсер антисейсмикалық жіктермен бөлінген әрбір блок үшін КМК 2.01.03-19 ұлттық нормасының спектрлік әдісімен анықталды. Нақты қаңқа 224,4 т болат жұмсайды, бұл жалпы аудан шаршы метріне 55,9 кг. Есептік базалық көлденең күш блок салмағының 3,7-4,5% құрайды, ал сервистік блоктың ең қолайсыз рамасының қабатаралық қисаюы 1/200 нормативтік шегінде 1/189 мәніне жетеді, сондықтан қаңқа пропорцияларын қатаңдық анықтайды. 5 іс-шара бағаланды: қималарды қайта қарау 33,8 т, иіліп дәнекерленген тұйық профильдерден жасалған торлы ригельдер 22,4 т, аражабын арқалықтарының болат-темірбетонды жұмысы 16,5 т, сервистік блоктың байланысты схемасы 8,4 т үнемдейді, болат класын С345 деңгейіне көтеру үнем бермейді. Іс-шаралардың келісілген үйлесімі шығынды 47,3 т немесе 21,1% азайтып, жалпы аудан шаршы метріне 44,1 кг деңгейіне жеткізеді. Нәтижелер қоңыржай сейсмикалы аудандардағы үлкен аралықты ғимараттар үшін қатаңдыққа бағдарланған шешімдердің басымдығын растайды.*

Түйін сөздер: *болат қаңқа, сейсмикалық төзімділік, спектрлік әдіс, қабатаралық қисаю, торлы ригель, болат-темірбетон арқалық.*

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НАУЧНАЯ СТАТЬЯ

СНИЖЕНИЕ МЕТАЛЛОЕМКОСТИ СТАЛЬНОГО КАРКАСА КРУПНОПРОЛЁТНОГО АВТОЦЕНТРА В СЕЙСМИЧЕСКОМ РАЙОНЕ

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Аннотация. *Стальные каркасы образуют основную часть стоимости несущих конструкций крупнопролётных коммерческих зданий в сейсмических районах, при этом источники избыточной металлоёмкости таких объектов редко документируются на данных реализованных проектов. Работа даёт количественную оценку резервов снижения расхода стали для 2-этажного автоцентра с бесколонным шоурумом пролётом 17 м, построенного в Ташкентской области на площадке сейсмичностью 7 баллов по шкале MSK-64. Плоская численная модель поперечной рамы реализована на языке Python, проверена 25 автоматическими верификационными тестами, сопоставлена с независимой конечноэлементной библиотекой с совпадением в пределах 0,01% и верифицирована по проектным огибающим усилий с отклонениями 7-12% по моментам и поперечным силам основных элементов и в пределах 13% по продольным силам колонн. Сейсмическое воздействие определено спектральным методом национальной нормы КМК 2.01.03-19 для каждого блока, отделённого антисейсмическими швами. Фактический каркас расходует 224,4 т стали, или 55,9 кг на квадратный метр общей площади. Расчётный базовый сдвиг составляет 3,7-4,5% веса блока, тогда как межэтажный перекос худшей рамы сервисного блока достигает 1/189 при нормативном пределе 1/200, поэтому пропорции каркаса определяются жёсткостью. Оценены 5 мероприятий: пересмотр сечений экономит 33,8 т, решётчатые ригели из гнутосварных замкнутых профилей 22,4 т, сталежелезобетонная работа балок перекрытия 16,5 т, связевая схема сервисного блока 8,4 т, повышение класса стали до С345 экономии не даёт. Согласованное сочетание мероприятий снижает расход на 47,3 т, или 21,1%, до 44,1 кг на квадратный метр. Результаты подтверждают приоритет жёсткостно-ориентированных решений для крупнопролётных зданий умеренной сейсмичности.*

Ключевые слова: *стальной каркас; сейсмостойкость; спектральный метод; межэтажный перекос; решётчатый ригель; сталежелезобетонная балка.*

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1 INTRODUCTION

The steel consumption of the frame determines the cost and the carbon footprint of a long-span building, which is why reducing the amount of steel while preserving earthquake resistance is now treated in design practice as a design task in its own right [1]. The scale of the effect has been quantified: for a two-storey steel-framed building an element-based assessment found that moving from non-seismic to high seismic design conditions raises the embodied carbon of the structure by about 51%, with the greatest relative rise falling on the beams and columns that provide the seismic stability [2].

Most publications on reducing the mass of steel frames deal with algorithmic optimisation. A formulation with uniform interstorey drift as the target condition reduced the mass of moment frames while fully utilising the code stiffness limit [3]. Optimisation of frames by the seismic collapse safety criterion confirmed that the effect persists when nonlinear behaviour is taken into account [4]. Strategies of discrete section sizing from product catalogues have been developed for space frames [5], metaheuristic algorithms have been compared for single-span pitched-roof portal frames [6], and the benefit of joint optimisation of geometry and sections has been shown for frames with tapered members [7]. Another group of reserves is formed by structural solutions. Weight minimisation of roof trusses has been studied with the effect of corrosion taken into account [8], and truss systems of long-span roofs have been compared by the criterion of material efficiency [9]. For steel-concrete composite beams an engineering approach to deflection design allowing for shear-connector slip has been proposed [10], and a comparison of the material costs of a steel beam and a composite beam showed the advantage of the latter [11]. Numerical studies of high-strength steel frames demonstrate that such systems are feasible in seismic regions when the governing constraints are strength-related [12].

The regional context is set by the updated probabilistic seismic hazard model for Central Asia [13]. Design practice in Uzbekistan relies on the response spectrum method of the code KMK 2.01.03-19 [14], whose application to multi-storey buildings is illustrated by the work of Tashkent authors [15]. Assessments of the earthquake resistance of framed buildings designed to codes of earlier generations underline the importance of revising structural solutions when the regulatory framework is updated [16].

The works listed above optimise notional test frames, whereas data on built structures with known design force envelopes appear in the open literature only in isolated cases. The author is not aware of any publication in which the steel consumption of a built long-span automobile centre at seismic intensity 7 has been analysed on the basis of a complete design bill of materials. The practical question of which requirement, strength or stiffness, limits the steel consumption of a moment-resisting scheme at moderate seismicity also remains open. The aim of the study is to quantify the reserves for reducing the steel consumption of the frame of a built automobile centre with a 17 m span in Tashkent region. The tasks include construction and verification of a numerical model against the design documentation, seismic analysis to KMK 2.01.03-19, identification of the active constraint, and assessment of five measures for reducing steel consumption and of their consistent combination.

2 MATERIALS AND METHODS

The object of the study is a two-storey automobile centre with a basement under part of the building, erected in Tashkent region on a site of seismic intensity 7 with category II soils. The overall dimensions between axes are 67.75 by 38.40 m, and the total floor area is 4014.4 m². Antiseismic joints divide the building into three blocks: a double-height showroom with a 17 m span without intermediate columns, a two-storey service block with a 3 by 6.0 m column grid, and a curved part. The column layout and the transverse frame are shown in Figure 1.

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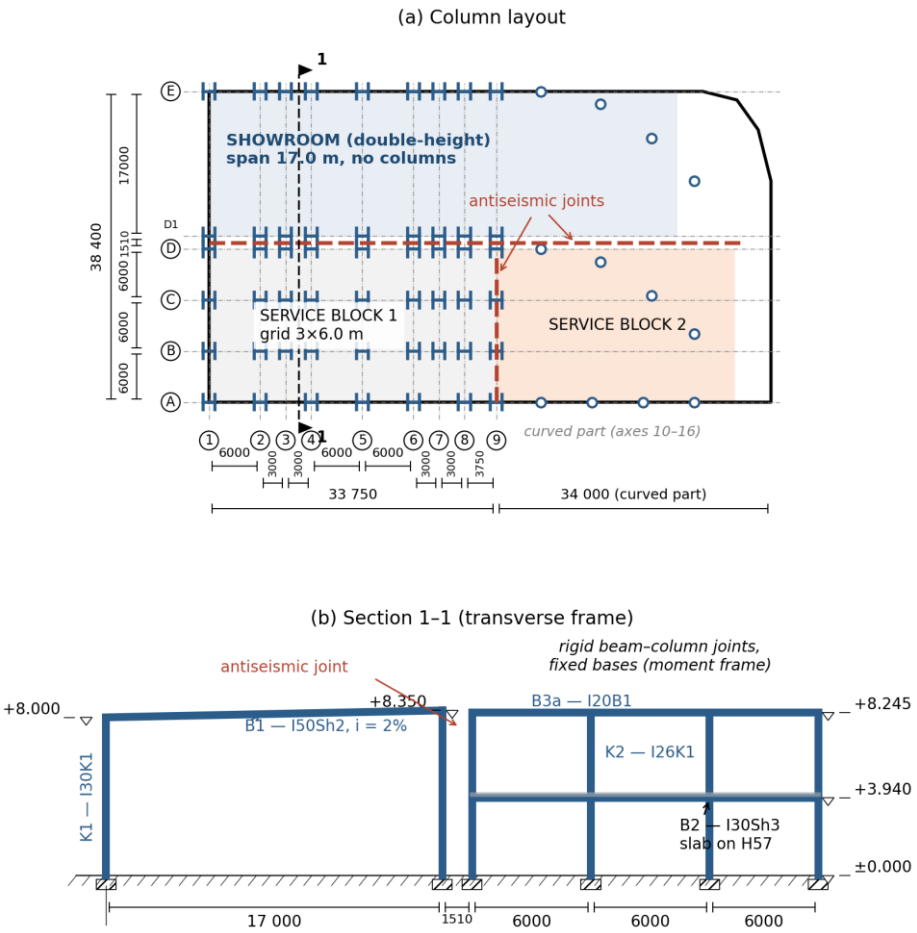

Figure 1 – (a) Column layout and (b) transverse frame of the building [author's materials]

The load-bearing system is a moment-resisting frame in both directions with fixed column bases and rigid beam-to-column joints; bracing and rigid diaphragms are provided in the roof. The floor at level +3.940 is a cast-in-place slab on H57-750-0.8 profiled steel decking, with 10 mm diameter shear studs welded to the beams through the decking. Steel grades S245 and S255 are used. Site splices of the beams are moved 600 mm away from the joint zone, and the connections of the secondary framing and sandwich panels have slotted holes, which isolates the non-structural elements from the frame drifts.

The loads are taken from the general data sheet of the structural design documentation and summarised in Table 1; the importance factor is taken as 0.95 for importance class II according to KMK 2.01.07-96 [17].

Table 1 – Loads and parameters of the construction site [author's materials]

Parameter	Value
Snow load, region I	0.50 kPa
Wind load, region I, terrain type A	0.38 kPa
Floor: permanent, design value	5.33 kPa
Floor: imposed, design value	2.40 kPa
Roof: permanent, design value	0.72 kPa
Roof: imposed, design value	1.40 kPa
Site seismic intensity, category II soils	7
Importance factor	0.95

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The transverse frame at section 1-1 was modelled as a plane bar system whose geometry follows the design drawings: the 17 m showroom portal and the service block with columns at 6.0 m centres carrying the floor beams at the +3.940 level and the roof beams at the +8.240 level. All members are two-node Euler-Bernoulli bar elements with three degrees of freedom per node, the axial and transverse displacements and the rotation, without shear deformation. The column bases are fixed and the beam-to-column joints are rigid, as adopted in the design; hinged member ends are represented by modified element stiffness matrices with the released rotation condensed out. The tributary width is 4.5 m for the showroom roof and 6.0 m for the worst frame of the service block, and the block-average value of 3.75 m is used for the block systems. A displacement-method solver was implemented in Python with the NumPy library by the direct stiffness algorithm: the element matrices are formed in local axes, rotated and assembled into the global matrix, the distributed loads are reduced to equivalent nodal forces through the fixed-end reactions, the support degrees of freedom are eliminated, the linear system is solved, and the member end forces, the span moment diagrams and the deflections are recovered from the nodal displacements. The input data, the loads, the geometry, the section database and the code coefficients, are held in plain-text modules, and every numerical result of the paper is generated from them by deterministic scripts, which makes the computation reproducible. Section sizing and checking were carried out to ShNK 2.03.05-13 [18] with a full enumeration of the catalogues of hot-rolled I-sections to GOST 26020-83 [19] and related profiles; the database of 86 items was cross-checked against two independent sources. The reliability of the model was assessed by comparison with the design envelopes from the member schedule, the availability of which for a built structure is a distinctive feature of the study. The codes KMK 2.01.07-96 and ShNK 2.03.05-13, in force when the building was designed, have since been replaced by GNP 2.01.07-21 and GNP 2.03.05-23; the design provisions on loads, deflection limits and design strengths of steel are given according to the editions used in the design.

The correctness of the pipeline was confirmed by an audit of 25 automatic checks in four groups, all of which pass. The solver group of 10 checks compares the model with closed-form solutions: a cantilever under a tip force reproduces the displacement $PL^3/3EI$ and the base moment to machine precision, a propped cantilever and a two-span continuous beam reproduce the support moment $qL^2/8$, the deflection of a simply supported beam matches $5qL^4/384EI$ within 0.1%, and a fixed-base portal under a distributed load satisfies vertical equilibrium together with the condition that the support and span moments of the girder sum to $qL^2/8$. The section group of 5 checks confirms the geometric consistency of all 86 catalogue items, the area against the plate dimensions within the rolling-radius allowance and the section modulus against the moment of inertia within 3%, and reconciles the group masses with the total of the bill of materials. The load group of 4 checks reassembles the design floor and roof loads of Table 1 from the layer composition within 10%. The code group of 6 checks verifies the implemented seismic functions: the property that the dissipation coefficient equals 1 at $\delta = 0.3$, the peak of 1.16 and the lower bound of 0.32 of the spectral curve, the continuity of the reduction factor between its branches, and the unit conversions. The solver was additionally cross-validated against the independent finite element library anastruct on the working schemes, with the discrepancy in bending moments and axial forces not exceeding 0.01%.

The plane idealisation corresponds to the structure of the building. The antiseismic joints divide it into three compact blocks, each with a regular column grid, roof bracing with rigid diaphragms and repeating transverse frames, so the transverse frame with its tributary strip represents the governing direction of the seismic response; the residual irregularity of the layout in plan is covered by the regularity factor of 1.1. Torsional effects, accidental eccentricities and the spatial interaction of the blocks lie outside the plane model, and the consequences of this simplification are discussed with the limitations of the study.

The seismic action was determined by the response spectrum method of KMK 2.01.03-19 [14] in the 2019 edition. The seismic force corresponding to the i-th mode of vibration at point k is calculated from formula (2.4) of the code, written as

$$S_{0ik} = \alpha \cdot Q_k \cdot W_i \cdot K_\delta \cdot \eta_{ik}, \quad (1)$$

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where α is the seismicity coefficient equal to 0.25 for site intensity 7 by Table 2.7 of the code, Q_k is the weight of the building assigned to point k of the design scheme, W_i is the spectral coefficient determined by clause 2.14 from the curve of region II for soils of categories I-II with the lower bound of 0.32, K_δ is the dissipation coefficient computed by the formula of clause 2.16 for the oscillation decrement δ = 0.15, which Table 2.9 assigns to frame buildings whose enclosure does not restrain their deformations, and η_{ik} is the mode shape coefficient determined by formula (2.6) of the code. The design force for the ultimate limit state follows formula (2.3) of the code and is obtained by multiplying S_{0ik} by the importance factor of Table 2.3, the recurrence factor of Table 2.4 and the storey-number factor of Table 2.10, each equal to 1.0 for the examined building, by the regularity factor of 1.1 assigned by Table 2.12 to the quasi-regular block layout, and by the reduction factor of clause 2.22 expressed by formula (2.11) of the code as

$$r = 0.85 \cdot \mu^{-0.67}, \tag{2}$$

where the permissible inelastic deformation factor μ reflects the ductility the code assigns to the structural system; the analysis adopts 15 for the steel moment-resisting blocks and 10 for the braced variant of the service block, corresponding to r values of 0.138 and 0.182. The blocks separated by joints were analysed separately: the showroom as a single-mass system and the service block as a two-mass system, with the fundamental periods determined by the Rayleigh method. Interstorey drifts for the serviceability limit state were calculated from the elastic seismic forces without reduction and compared with the deformation limits of clause 2.12 and Table 2.6 of KMK 2.01.03-19: 1/200 of the storey height for buildings whose brittle non-structural elements are attached to the load-bearing structures and 1/70 for buildings whose non-structural elements are isolated from the deformations of the frame.

The measures for reducing steel consumption were assessed by sizing the sections to the design envelopes with simultaneous checks of strength with a working condition factor of 1.1 for seismic combinations, shear, deflections to the stiffness requirements of KMK 2.01.07-96, column buckling to curve b, limiting slenderness and drifts. Five measures were considered: resizing of sections within the design scheme, conversion of the service block to a braced scheme, replacement of the solid-web showroom beams by trusses of cold-formed welded hollow sections, raising the steel grade of the beams to S345, and allowing for composite action of the floor beams with the shear studs already provided.

3 RESULTS AND DISCUSSION

According to the total line of the bill of materials, the frame consumes 224.4 t of steel; the sum of the individual lines reaches 228.13 t because of two items of cold-formed profiles not included in the total, and the design total value is used in this work. The specific consumption is 55.9 kg/m² of the total floor area and 83.3 kg/m² of the footprint area. The structure of consumption is shown in Figure 2: beams account for 143.06 t, or 63.8%, columns 65.05 t, or 29.0%, purlins 9.72 t and bracing 6.57 t. The largest items are the secondary floor beams of 23Sh1 I-section with a mass of 43.4 t, the showroom beams of 50Sh2 with a mass of 31.2 t, and the plate details of joints and bases with a mass of 40.35 t.

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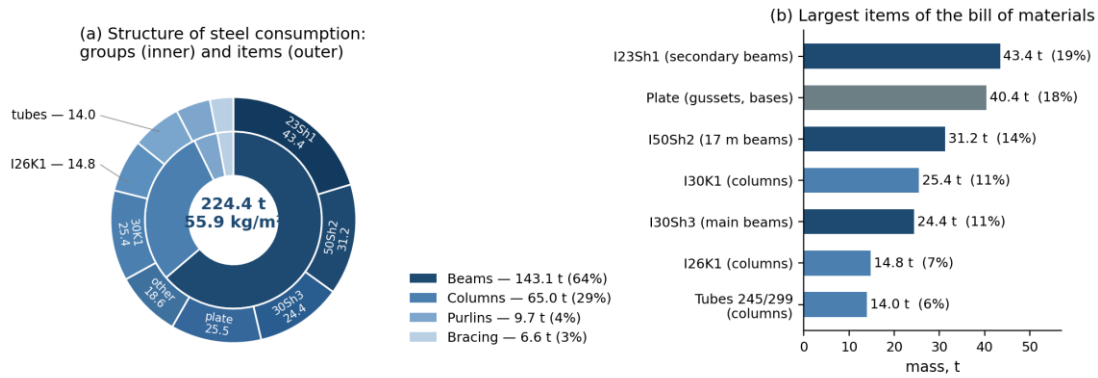

Figure 2 – Structure of the steel consumption of the frame (a) and the largest items of the bill of materials (b) [author's materials]

The results of the model verification are given in Table 2 and Figure 3. The bending moments and shear forces of the main members deviate from the design envelope by 7-12%, and the axial forces of the columns, collected from tributary areas, agree within 13%. The only exception is the light roof beam B3a with a deviation of +18.6%. The forces are given in the tonne-force units of the design documentation with SI equivalents in parentheses; 1 tf equals 9.81 kN.

Table 2 – Comparison of the model forces with the design envelope [author's materials]

Member, force	Model	Design	Deviation
B1, I-section 50Sh2: M, tf·m (kN·m)	28.12 (275.8)	31.60 (309.9)	−11.0%
B1: Q, tf (kN)	9.75 (95.6)	10.68 (104.7)	−8.7%
K1, I-section 30K1: M, tf·m (kN·m)	14.72 (144.4)	16.72 (164.0)	−11.9%
B2, I-section 30Sh3: M, tf·m (kN·m)	16.41 (160.9)	17.72 (173.8)	−7.4%
B2: Q, tf (kN)	15.84 (155.3)	14.27 (139.9)	+11.0%
B3a, I-section 20B1: M, tf·m (kN·m)	4.23 (41.5)	3.57 (35.0)	+18.6%
N of column K1, tf (kN)	31.5 (308.9)	27.8 (272.6)	+13%
N of column K2, tf (kN)	22.2 (217.7)	22.0 (215.7)	+1%
N of column K3, tf (kN)	24.3 (238.3)	25.1 (246.2)	−3%
N of column K4, tf (kN)	31.4 (307.9)	32.4 (317.8)	−3%

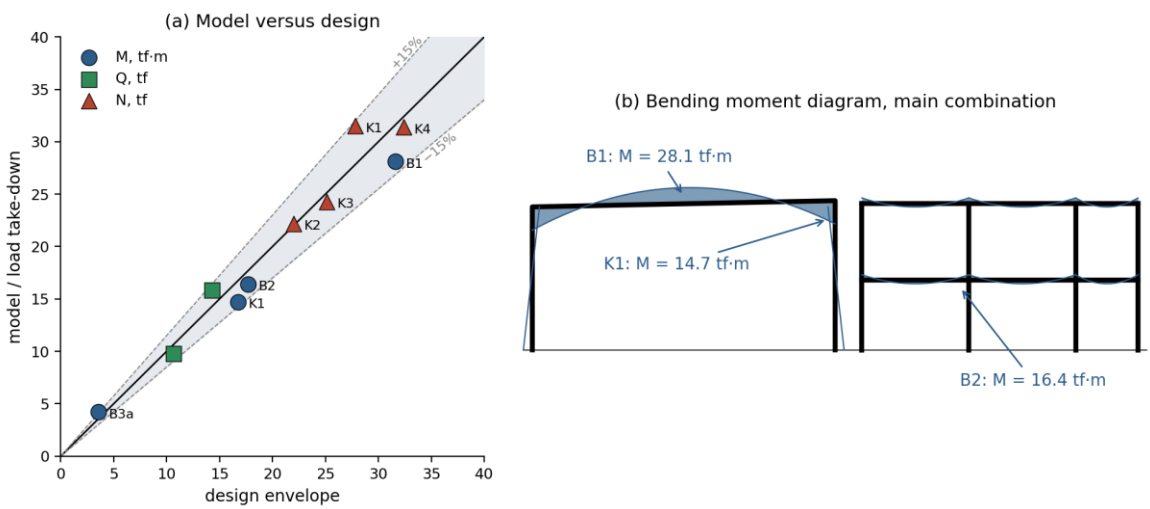

Figure 3 – Verification of the model against the design force envelopes [author's materials]

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The 18.6% deviation for the light beam B3a is explained by the envelope nature of the design values over all members of the mark and by local snow drifts at the skylights, which in the model are distributed along the span. The systematic underestimation of the roof moments has the same origin. The deflection of beam B1 under the full characteristic load was 39.4 mm against the limit of 62.8 mm. The accuracy achieved is sufficient for a comparative analysis of variants of the same scheme.

The results of the seismic analysis are summarised in Table 3 and Figure 4. The fundamental periods of the blocks were 0.55 s for the showroom and 0.49 s for the service block; for the worst end frame of the service block with a 6 m tributary strip the period reaches 0.61 s. The design base shear for the ultimate limit state equals 3.7-4.5% of the block weight, which is explained by the deep reduction of the spectral forces for ductile steel frames.

Table 3 – Results of the seismic analysis of the blocks [author's materials]

Block, design case	T_1, s	Shear, % of weight	Drift	Limit
Showroom, block system	0.55	4.4	1/373	1/200
Service, storey 1, block system	0.49	4.5	1/260	1/200
Service, storey 2, block system	0.49	4.5	1/293	1/200
Service, storey 1, end frame	0.61	3.9	1/189	1/200 and 1/70
Service, storey 2, end frame	0.61	3.9	1/216	1/200 and 1/70

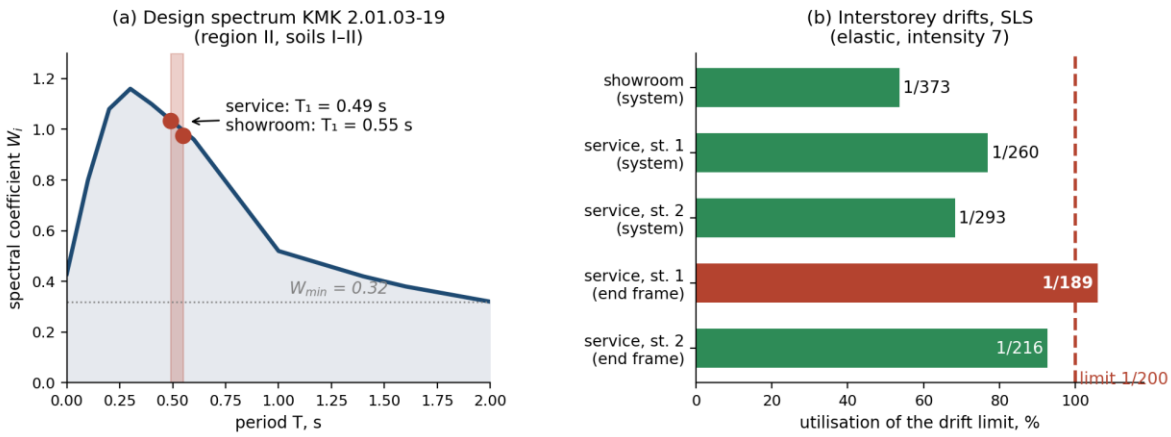

Figure 4 – Design spectrum (a) and utilisation of the drift limit (b) [author's materials]

The strength utilisation of the sections under the design envelopes lies in the range 0.50-0.75, and the deflection of beam B1 reaches 63% of the limit. The picture for stiffness is different: the drifts of the block systems are 1/373 and 1/260 against the limit of 1/200, whereas the end frame of the first storey of the service block reaches 1/189 and exceeds the limit by 6%. Table 2.6 of KMK 2.01.03-19 allows such a drift under the 1/70 limit for buildings whose non-structural elements are isolated from the deformations of the frame; in the examined building the sandwich panels and the secondary framing are attached through slotted holes, which is the isolation measure this provision presumes, and its confirmation for every non-structural element belongs to the detailed design. The governing constraint of the moment-resisting scheme at intensity 7 is stiffness, while a strength reserve is retained. This conclusion agrees with the results of moment frame optimisation, in which the active condition turns out to be equality of the drifts to the code limit [3].

The smallness of the strength-level seismic forces admits a direct decomposition. The product of the seismicity coefficient 0.25 and the reduction factor 0.138 gives 0.035 of the weight; multiplication by the spectral coefficient of about 1.0 at periods of 0.49-0.61 s and by the regularity factor 1.1 leads to 3.8% of the weight, which matches the data of Table 3. The decomposition shows

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that the decisive contribution to the smallness of the forces comes from the reduction, which reflects the plastic reserve of steel frames, whereas the spectral ordinate itself at these periods is close to the maximum of the curve. The conclusion is robust to the choice of the dissipation parameter: the adopted value of 0.15 is conservative, moving to 0.3 would reduce the seismic forces by 14%, and tightening in the opposite direction would raise them by the same amount; for any of these values the forces remain within 5% of the weight and the strength checks retain a margin.

The assessment of the measures for reducing steel consumption is given in Table 4, their effect and the consistent combination are shown in Figure 5, and the structural schemes in Figure 6.

Table 4 – Measures for reducing steel consumption and their effect [author's materials]

Variant	Content	Saving, t
V1	Resizing of sections within the design scheme: 50Sh2→55B1, 30Sh3→35B2, 23Sh1→25B1, purlins→12B1	33.8
V2	Braced service block: 26K1→20K1, bracing +1.84 t, pinned beams 40B2	8.4
V3	Trusses of cold-formed hollow sections on the 17 m span, h = 1.2 m	22.4
V4	Steel S345 for beams	≈0
V5	Composite floor beams: 23Sh1→20B1	16.5
V*	Consistent combination V3 + V5 + V2 + elements of V1	47.3

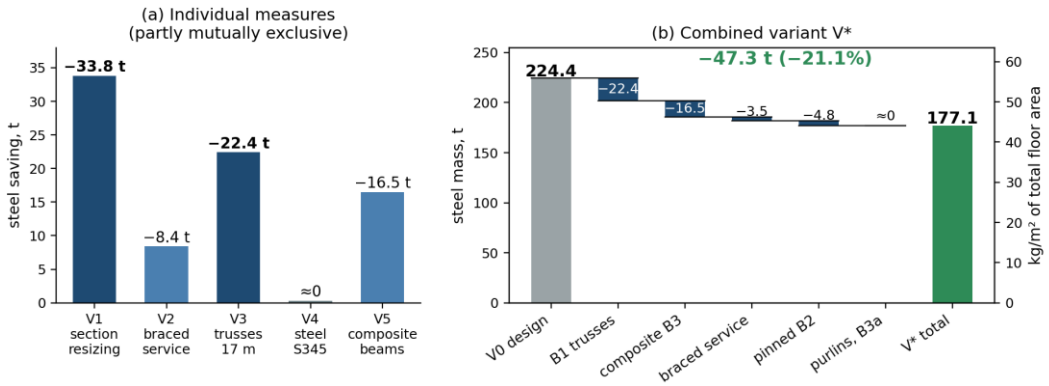

Figure 5 – Effect of the individual measures and the combined variant [author's materials]

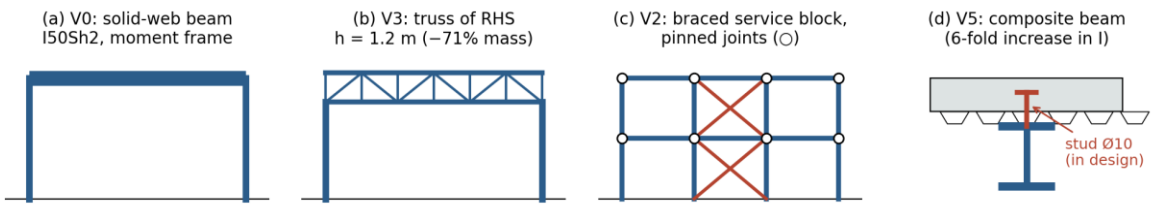

Figure 6 – Structural schemes of the variants [author's materials]

The first measure V1 retains the design scheme and replaces the sections by the nearest sufficient ones: the 50Sh2 beam is replaced by a 55B1 I-section, the 30Sh3 main beam by 35B2, the 23Sh1 secondary beam by 25B1, and the channel-type purlin by a 12B1 profile; the total saving is 33.8 t, or 15.1%. The second measure V2 converts the service block to a braced scheme: the 26K1 columns are replaced by 20K1 while keeping the I-section as required by clause 3.10 of KMK 2.01.03-19, vertical bracing with a mass of 1.84 t is added, and the beams are pin-connected and increased to 40B2; the resulting saving is 8.4 t with the drifts having a margin to the limit.

The third measure V3 replaces the solid-web beams of the 17 m span by 1.2 m deep trusses with 12 panels; chords of 100×100×4 profile and web members of 80×80×3 give a mass per unit

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length of 39 kg/m against 138.7 kg/m for the solid section, the truss deflection is 53 mm against the limit of 62.8 mm, and the saving reaches 22.4 t, or 71% of the group mass. The 39 kg/m of the lattice girder is a complete value that includes an 18% allowance for the node details and gusset plates of the truss, while the building-level group of plate details, 40.35 t, was kept at the as-built mass in all variants, so the quoted saving compares complete assemblies and remains a lower-bound estimate. The fourth measure V4 raises the steel grade of the beams to S345 and brings no saving, since the sections are limited by deflections, which do not depend on the design strength. This negative result confirms the stiffness-related nature of the constraints and agrees with the data on the rational field of application of high-strength steels in members governed by strength [12].

The fifth measure V5 allows for the composite action of the floor beams with the cast-in-place slab through the 10 mm shear studs provided in the design. The transformed section was computed for a 6.0 m beam on a 1.5 m tributary strip with the effective slab width taken as the smaller of the beam spacing and a quarter of the span, a 120 mm solid slab with the ribbed part over the H57 decking neglected, and a modular ratio of 6.9 for the B25 concrete. For the 20B1 section the effective moment of inertia grows from 1943 to 11649 cm^4, six times the steel value, and the deflection under the characteristic load drops to 7 mm against the limit of 30 mm; the erection stage was checked separately for the steel section alone under the fresh concrete and an erection load of 0.75 kPa. The 23Sh1 beam is replaced by 20B1 with a saving of 16.5 t. The engineering treatment of shear-connector slip in such an analysis is described in [10].

The efficiency of the trusses has a clear mechanical explanation. The 1.2 m depth of the lattice girder against 0.5 m for the solid I-section increases the lever arm of the internal couple 2.4-fold, while the chord material works in axial force with full utilisation of the section. The truss deflection of 53 mm against the limit of 62.8 mm is telling: the utilisation of the stiffness limit reaches 84%, so the lattice solution also remains stiffness-governed, and further lightening of the chords runs up against deflection, which sets a lower bound on the mass per unit length of about 39 kg/m for this span and load.

Conversion of the service block to a braced scheme is accompanied by an increase in the design seismic forces, since the reduction factor of braced systems, 0.182, exceeds that of moment frames, 0.138, by 32%; nevertheless the resulting mass decreases, because the columns are relieved of bending and work in compression, while the drift is resisted by bracing with high axial stiffness. The combination of increased forces with reduced mass shows that at moderate seismicity the choice of the structural scheme should be made by the criteria of stiffness and steel consumption, with the difference in seismic forces remaining a secondary factor.

The deep reduction of the seismic forces presumes the inelastic deformation capacity that KMK 2.01.03-19 assigns to steel frames through the factor μ of formula (2), and this assumption imposes requirements on the proposed measures. The trusses of V3 and the composite beams of V5 remain within the moment-resisting blocks analysed with $\mu = 15$, and their strength utilisation under the design combinations stays below 0.75, so the demand for inelastic work at intensity 7 is low. The braced scheme of V2 is analysed with the lower value $\mu = 10$ and correspondingly higher forces, and its columns keep the I-section required by clause 3.10 of the code. The capacity of the connections, the welded rigid joints, the truss nodes and the bracing gussets, to develop the assumed plastic mechanisms is a matter of detailing and is addressed in the limitations.

The negative result for steel S345 can be generalised as an engineering criterion: raising the steel grade is justified when the strength utilisation of the sections is close to 1 and there is a stiffness reserve. In the frame considered the ratio is the opposite: the strength utilisation is 0.50-0.75, whereas the utilisation of the drift limit reaches 106%. The plate details remain an untapped reserve: their 40.35 t make up 18% of the consumption, and a braced scheme with pinned joints reduces the number of welded rigid connections and simplifies the gusset plates, so the estimated effect of the combination should be regarded as a lower bound.

The consistent combination of measures without double counting includes trusses on the 17 m span, composite floor beams, a braced service block and elements of section resizing. The steel consumption decreases from 224.4 to 177.1 t, that is by 47.3 t, or 21.1%, and the specific indicator

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falls from 55.9 to 44.1 kg/m² of the total floor area. The value obtained is comparable with the effects of algorithmic optimisation of notional frames, for which mass reductions of 10-25% are reported [4, 5], and exceeds the effect of section resizing within an unchanged scheme. The structure of the combination underlines the main conclusion of the work: 38.9 t of the 47.3 t, or 82% of the total saving, come from solutions that increase stiffness per unit mass, namely trusses and composite action, whereas replacing sections with the structural form unchanged realises the residual reserve. For the design practice of similar buildings this implies an order of priorities: first the choice of the structural form of the long-span beams and of the scheme resisting the drifts, then allowance for the composite action of steel with the slab, and only after that refinement of the catalogue items.

The results also invite a comparison with work published in the Bulletin of the Kazakh Leading Academy of Architecture and Civil Engineering, where the reduction of steel and labour consumption in framed structures forms an established research line. For welded crane runway beams a corrugated web lowers the beam mass by 4.9-6.9% and the fabrication labour by 20-40% [20], raises the out-of-plane inertia of the web by two orders of magnitude with a steel saving of up to 20% [21], and tolerates framed technological openings of up to half the web depth, with paired stiffeners restoring 20-25% of the lost capacity [22]. These studies exploit the same physical idea as the lattice girders of the present work, the removal of the least stressed web material, and the difference lies in the scale: refining the web saves 7-20% of the element mass, whereas changing the structural form of the 17 m girders releases 71% of the group mass.

The journal's analysis of a five-storey reinforced concrete frame in a seismic zone, where a regular scheme with distributed shear walls shortened the fundamental period from 0.839 to 0.444 s and cut the reinforcement demand by 28%, reaches the same conclusion as the present study: the rational scheme is at once safer and more economical [23]. The survey of seismic resistance theory in the same journal notes the limitations of the response spectrum method for tall buildings [24]; for the two-storey building considered here, divided into regular blocks, the method remains adequate, and its conventionality is offset by the verification of the model against the design envelopes. The composite reserve of the floor beams has a counterpart in concrete-filled steel tubular arches, where the joint work of steel and concrete cuts the material demand of small arches by 30-40% [25], and geotechnical seismic isolation, which reduces the accelerations transmitted to the building by 19-29% at a cost below that of structural isolation systems [26], acts on the demand side and can complement the capacity-side measures examined here.

A comparison with international studies of the optimisation of seismic-resistant steel frames supports the same hierarchy of reserves. In drift-driven optimisation of moment frames the active constraint is the equality of the interstorey drifts to the code limit [3], which matches the state of the examined frame, where the worst drift reaches 106% of the general limit while the strength utilisation stays at 0.50-0.75. Metaheuristic sizing of portal frames [6] and the joint optimisation of the geometry and sections of tapered members [7] show that adding the geometry of the structure to the design variables increases the achievable saving beyond pure section selection, in agreement with the 82% share of the form-changing measures in the total saving obtained here. The absence of a benefit from the S345 grade in the stiffness-governed scheme is consistent with numerical studies of high-strength steel moment frames, which place their rational field where the governing constraints are strength-related [12].

The limitations of the work define its domain of applicability. The quantitative results refer to a two-storey steel building of regular block layout with spans up to 17 m, erected on category II soils at site intensity 7 and designed to the load levels of Table 1; extension to taller buildings, higher intensities, softer soils or irregular layouts requires a new analysis. The calculations were carried out on plane models of the transverse frames against the design force envelopes: torsional effects, accidental eccentricities and the spatial interaction of the blocks are covered only by the regularity factor of the code, so the proposed solutions are a feasibility assessment, and their implementation requires verification on a spatial model of the building together with a check of the joints, including the capacity of the connections to develop the ductility assumed by the reduction factor. The plate details with a mass of 40.35 t were kept unchanged when the variants were recalculated, and the

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braced scheme simplifies the joints, so the actual effect of the combination is higher than calculated. The depth of the new beam sections exceeds the original by 5-10 cm, which is acceptable for the double-height showroom and for ceilings 3.0-3.15 m high. The applicability of the 1/70 drift limit rests on the isolation of the non-structural elements, which the working design provides through slotted connections and which must be confirmed for every element at the detailed design stage. Trusses increase the labour input for the joints and the area of fire protection; the cost side of the solutions was not assessed in this work and forms the subject of a separate study.

4 CONCLUSIONS

1. The actual steel consumption of the frame of the built two-storey automobile centre with a 17 m span at seismic intensity 7 was 224.4 t, or 55.9 kg/m² of the total floor area; the dominant consumption group is formed by beams with a share of 63.8%.

2. The model, verified against the design envelopes with deviations of 7-12% for the moments and shears of the governing members and within 13% for the column axial forces, showed that the design seismic forces amount to 3.7-4.5% of the block weight, whereas the interstorey drift of the worst frame reaches 1/189 against the limit of 1/200; the governing constraint of the moment-resisting scheme is stiffness.

3. The largest single reserve is provided by replacing the solid-web beams of the 17 m span by trusses of cold-formed hollow sections with a saving of 22.4 t, or 71% of the group mass; allowing for the composite action of the floor beams with the shear studs provided in the design adds 16.5 t.

4. Raising the steel grade to S345 in a stiffness-governed scheme brings no saving, since the limiting deflections and drifts do not depend on the design strength of the steel.

5. The consistent combination of measures reduces the steel consumption by 47.3 t, or 21.1%, to 44.1 kg/m², while satisfying all the strength, stiffness and seismic checks of the applicable codes.

6. A further direction of research is cost optimisation taking into account the labour input for joints, fire protection and erection.

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