

UDC 624.074
IRSTI 67.11.31
RESEARCH ARTICLE

EXPERIMENTAL VERIFICATION OF THE STRAIN-STRESS DISTRIBUTION OF A HINGED VENTILATED FACE SUBSYSTEM UNDER NONSTATIONARY LOADING

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Abstract. *Hinged ventilated faces (HVF) are among the most effective solutions for thermal renovation of buildings in extreme continental climates. A previous study developed a “boundary optimization” methodology for the HVF subsystem topology using the finite-element technique (FET): increasing the bracket spacing from 600 × 600 mm to 1000 × 1000 mm reduces the thermal irregularity coefficient from 8.33% to 3.0% while keeping stresses within 75% of the material’s yield point. However, experimental verification under seismic loading has been lacking, limiting the approach’s direct application in design practice. This study experimentally verifies the finite element model of an HVF subsystem under nonstationary loading, for bracket radiuses of 150 mm and 200 mm at 600 × 600 mm spacing. The novelty lies in the quantitative justification of the FE modeling strategy through direct comparison of calculated and experimental von Mises stresses and displacements, with a physical explanation of their systematic discrepancy. Tests were performed on full-scale (1:1) specimens using a uniaxial electrodynamic vibration bed, with seismic action defined by synthetic accelerograms compatible with design spectra for soil category III at seismicity of 8 and 9 (MSK-64). Deformations were measured with strain gauges and displacements with laser sensors, for four configurations under two seismic directions. The resulting deviation of 12.0-12.6% remained within the ±15% verification threshold, confirming the FE strategy’s adequacy for engineering calculations and providing an experimentally validated basis for extending the “boundary optimization” methodology to larger bracket spacings and other seismically active regions.*

Keywords: *boundary optimization; finite element modeling; von Mises stress; bracket spacing; seismic testing; thermal bridging; full-scale testing.*

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ӘОЖ 624.074
ҒТАМР 67.11.31
ҒЫЛЫМИ МАҚАЛА

ІЛМЕЛІ ЖЕЛДЕТІЛЕТІН ҚАСБЕТ ІШКІ ЖҮЙЕСІНІҢ СТАЦИОНАРЛЫҚ ЕМЕС ЖҮКТЕМЕ КЕЗІНДЕГІ КЕРНЕУ- ДЕФОРМАЦИЯЛЫҚ КҮЙ ТАРАЛУЫН ЭКСПЕРИМЕНТТІК ВЕРИФИКАЦИЯЛАУ

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Аңдатпа. *Континенттік климаты өткір аймақтардағы ғимараттарды жылу жағынан жаңғыртудың тиімді жолдарының бірі - ілмелі желдетілетін қасбеттер (ІЖҚ) жүйесі. Авторлар бұған дейін ақырлы элементтер әдісіне (АЭӘ) негізделген «шекаралық оңтайландыру» әдістемесін ұсынған болатын: кронштейндер қадамын 600 × 600 мм-ден 1000 × 1000 мм-ге дейін ұлғайту жылу біртексіздігі коэффициентін 8,33%-дан 3,0%-ға дейін азайтады, ал кернеулер материалдың аққыштық шегінің 75%-ынан аспайды. Дегенмен бұл нәтижелер сейсмикалық жүктеме жағдайында тексерілмегендіктен, әдістеменің қолданылу аясы шектеулі болды. Осы зерттеуде кронштейн радиусы 150 және 200 мм, қадамы 600 × 600 мм болатын ІЖҚ ішкі жүйесінің ақырлы элементтер моделі стационарлық емес жүктеме жағдайында эксперименттік тексерілді. Ғылыми жаңалығы - есептік және эксперименттік фон Мизес кернеулері мен ығысуларды тікелей салыстыру арқылы АЭӘ моделдеу стратегиясын сандық негіздеу және олардың жүйелі айырмашылығының физикалық себебін түсіндіру. Сынақтар бір осьті электродинамикалық тербеліс стендінде, толық ауқымды (1:1) үлгілерде жүргізілді; сейсмикалық әсер III санатты топырақ үшін МСК-64 шкаласы бойынша 8 және 9 балға сәйкес акселерограммалармен берілді. Деформациялар тензодатчиктермен, ығысулар лазерлік датчиктермен төрт конфигурация үшін екі бағытта өлшенді. Алынған 12,0-12,6% ауытқу ±15% верификациялық шектен аспайды, бұл АЭӘ стратегиясының инженерлік есептеулер үшін жеткілікті дәлдігін растайды және «шекаралық оңтайландыру» әдістемесін кронштейндер қадамының кеңейтілген диапазонына (800 × 800 және 1000 × 1000 мм) және басқа сейсмикалық белсенді өңірлерге қолдануға негіз қалыптастырады.*

Түйінді сөздер: *шекаралық оңтайландыру; ақырлы элементтер әдісі; фон Мизес кернеуі; кронштейн қадамы; сейсмикалық сынақ; жылу көпірлері; толық ауқымды сынақ.*

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УДК 624.074
МРНТИ 67.11.31
НАУЧНАЯ СТАТЬЯ

ЭКСПЕРИМЕНТАЛЬНАЯ ВЕРИФИКАЦИЯ НАПРЯЖЕННО-ДЕФОРМИРОВАННОГО СОСТОЯНИЯ ПОДСИСТЕМЫ НАВЕСНОГО ВЕНТИЛИРУЕМОГО ФАСАДА ПРИ ДИНАМИЧЕСКОМ НАГРУЖЕНИИ

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Аннотация. Навесные вентилируемые фасады (НВФ) - один из наиболее действенных способов тепловой защиты зданий в условиях резко континентального климата. Ранее авторами предложена методика «граничной оптимизации» топологии подсистемы НВФ на основе метода конечных элементов (МКЭ): увеличение шага кронштейнов с 600 × 600 мм до 1000 × 1000 мм снижает коэффициент тепловой неравномерности с 8,33 % до 3,0 % при напряжениях не более 75 % предела текучести материала. Однако экспериментальная проверка этих прогнозов при сейсмических нагрузках отсутствовала, что ограничивало применение методики в проектной практике. В настоящей работе выполнена экспериментальная верификация КЭ-модели подсистемы НВФ при динамическом нагружении для вылетов кронштейна 150 и 200 мм при шаге 600 × 600 мм. Научная новизна состоит в количественном обосновании стратегии КЭ-моделирования путём прямого сопоставления расчётных и экспериментальных эквивалентных напряжений по Мизесу и перемещений, с физическим объяснением их систематического расхождения. Испытания проводились на натурных образцах (1:1) на одноосном электродинамическом вибростенде; сейсмическое воздействие задавалось синтетическими акселерограммами для грунта III категории при сейсмичности 8 и 9 баллов (МСК-64). Деформации измерялись тензорезисторами, перемещения - лазерными датчиками, для четырёх конфигураций в двух направлениях. Полученное расхождение 12,0-12,6 % укладывается в порог верификации ±15 %, подтверждая пригодность стратегии КЭ-моделирования для инженерных расчётов и создавая экспериментально обоснованную базу для распространения методологии «граничной оптимизации» на увеличенные шаги кронштейнов (800 × 800 и 1000 × 1000 мм) и другие сейсмически активные регионы.

Ключевые слова: граничная оптимизация; метод конечных элементов; напряжения по Мизесу; шаг кронштейнов; сейсмические испытания; тепловые мосты; натурные испытания.

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1 INTRODUCTION

A key challenge in the design of hinged ventilated faces is ensuring a rational balance between energy efficiency [1, 2] and structural reliability [3]. Metal fastening elements, such as brackets and guiding profiles, form heat bridges that reduce the overall thermal resistance of the wall [4, 5]. In addition to that, these elements must reliably transfer static, wind, seismic, and icing loads to the load-bearing structure [6, 7]. Thus, optimizing the subsystem geometry, and primarily the bracket spacing, is a problem solved simultaneously in the thermal and structural planes.

In a previous parametric numerical study [2], a “boundary optimization” methodology for the topology of the hinged ventilated face subsystem was developed. Using the finite-element technique (FET), it was shown that increasing the bracket spacing from 600×600 mm to 1000×1000 mm reduces the thermal irregularity coefficient from approximately 8.33% to 3.0% while maintaining equivalent stresses within 75% of the material’s yield point even under the most unfavorable load combinations for five representative climatic zones of the Republic of Kazakhstan [2, 8-10]. However, the reliability of FET predictions for such systems under seismic loading, in the absence of experimental data, remains insufficiently substantiated, which hinders the direct application of the obtained results in design practice.

The importance of experimental verification of face structures’ numerical models is well known in the scientific literature. A number of studies devoted to the mechanical strength of face elements under the influence of working loads [11-13] have found that the nature of the applied loads directly affects the degree of structural irregularity and the formation of localized zones of increased thermal losses. The static behavior of face systems was studied in [14], in which load-bearing profiles, fastening elements and the interaction of cladding panels with the building frame were analyzed. The field tests and numerical calculations in these works demonstrated the critical role of correct design of transverse and longitudinal elements to ensure stiffness and prevent plastic deformation. The impact of wind loads and their consequences, i.e. wind vibrations, local rarefaction zones in the ventilated gap, fatigue damage in fastening points, were studied in [15-17], which emphasized the role of aerodynamic modeling and field tests. Works [18-20] are devoted to the seismic behavior of modular face systems. The results obtained indicate the positive effect of flexible connections and deformable joints on energy dissipation and resistance to element separation [21-22]. The combined effects of icing, temperature cycles and wind-driven rain create additional risks: ice growth, changes in the size of gaps in joints and disruption of aerodynamic characteristics [23].

However, virtually all existing experimental studies consider the structural and thermal characteristics of face systems separately. An integrated approach that combines experimental verification of the structural response with subsequent assessment of the thermal irregularity coefficient under conditions characteristic of the natural and climatic environment of a particular region remains underdeveloped. In particular, experimental data confirming the adequacy of finite element models for HVFs under seismic impacts corresponding to highly seismic zones of Kazakhstan (7-9 points MSC-64) are virtually absent in the published literature. The methodology of the standard [24], which forms the basis for the numerical assessment of heat-conducting inclusions, regulates the requirements for calculating heat flows and surface temperatures, but does not establish requirements for the experimental verification of structural models under combined working loads.

In connection with the above, the aim of this study is the experimental verification of the finite element model of the hinged ventilated face subsystem under seismic loading, performed for the climatic zones of series B (Shymkent / Ust-Kamenogorsk, seismicity of 8 points MSK-64) and series E (Almaty, seismicity of 9 points MSK-64), with bracket extensions of 150 mm and 200 mm and a bracket spacing of 600×600 mm [2]. The scientific novelty of the work lies in the quantitative justification of the adequacy of the adopted FE modeling strategy through a direct comparison of the calculated and experimentally measured equivalent von Mises stresses and displacements, as well as in establishing the physical nature of the systematic discrepancy between the numerical and experimental results, caused by the difference in the composition of the load combinations taken into account.

The scope of the study covers the design and reconstruction of multistorey buildings in seismically active regions with a continental climate. The practical significance of the work lies in the verified confirmation that the FET-based “boundary optimization” methodology developed in the previous parametric study is adequate for extension to larger bracket spacings (800×800 mm and 1000×1000 mm) and to the full range of climatic series [2], including regions in which the design load combination is determined by seismic action.

2 MATERIALS AND METHODS

2.1 Experimental Verification Program

To validate the parametric finite element analysis results, a series of physical tests was conducted on full-scale specimens of the hinged ventilated face subsystem. The test program was designed to reproduce loading conditions corresponding to Series B (Shymkent / Ust-Kamenogorsk climate zone: seismicity of 8 points MSC-64, wind region III, soil category III, ice region III) and Series E (Almaty climate zone: seismicity of 9 points MSC-64, wind region II, soil category III, ice region III), with bracket extension lengths of 150 mm and 200 mm and a bracket spacing of 600×600 mm for the following load combinations (Fig. 1) [2]:

- (i) Seismic action along the X-axis;
- (ii) Seismic action along the Y-axis.

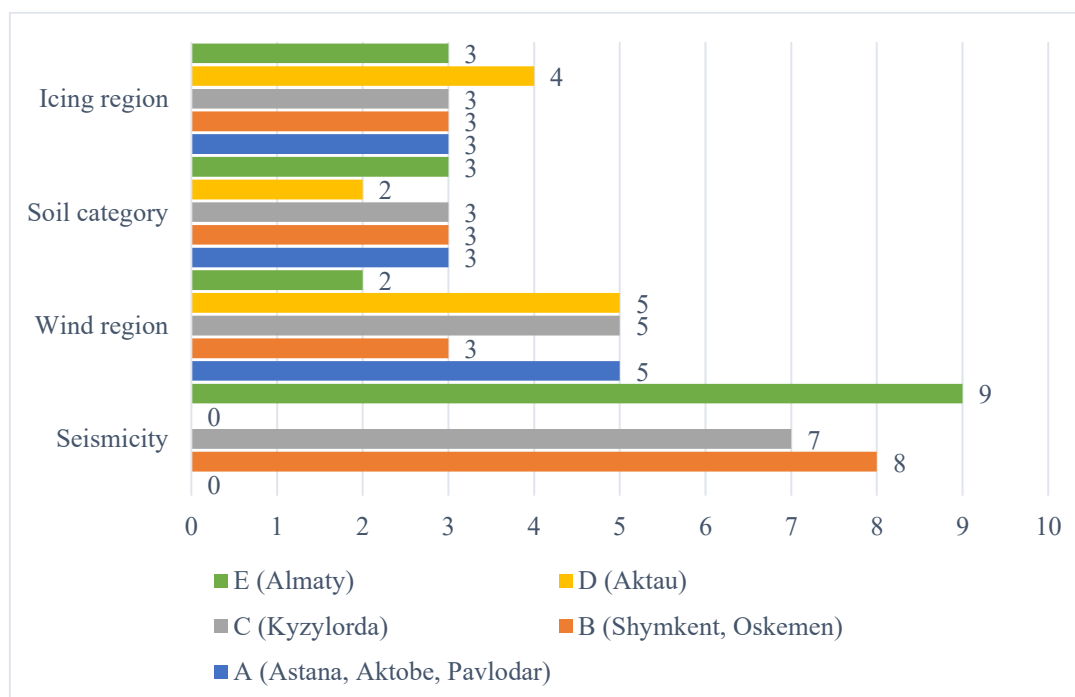


Figure 1 – Series of analysis of working loads of a hinged face system [2]

It should be pointed out that ice accretion on the structure was not modeled in this experimental setup due to technical limitations of the test equipment. Accordingly, the calculated values from Tables 8 and 9 of the previous parametric study [2], which include the ice load component, serve as upper estimates when comparing results.

2.2 Test Specimens and Setup

The test specimens were 1:1 scale fragments of the hinged ventilated face system (Fig. 2) made of the same materials as the numerical model: galvanized steel grade St08ps (bracket sheet thickness 2 mm, T-shaped guiding profile thickness 1.5 mm). Tests were conducted for two bracket extension configurations, 150 mm and 200 mm, with a bracket spacing of 600×600 mm. Ceramic granite with

a density of 2400 kg/m^3 was used as the cladding panels, secured to the subsystem with standard clamps. The total mass of the specimen, including the cladding and subsystem, was approximately 85 kg (150 mm extension) and approximately 88 kg (200 mm extension).

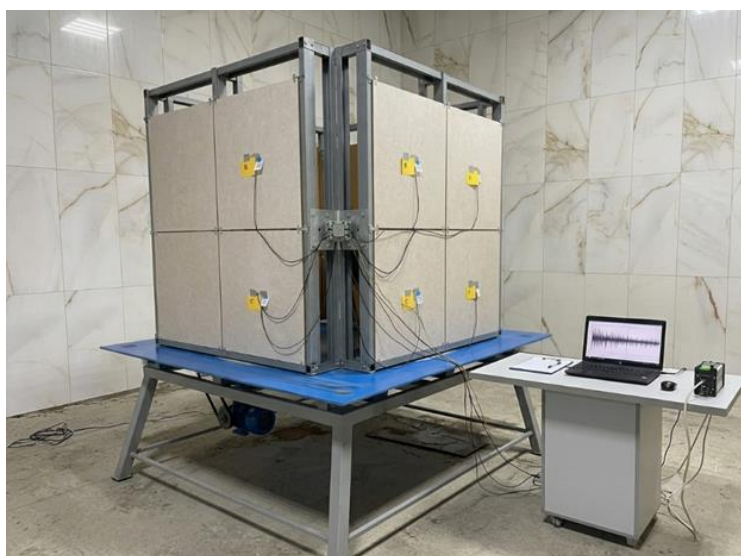


Figure 2 – Schematic diagram of the test setup and HVF specimen:
the specimen mounted on the platform of a uniaxial electrodynamic vibration bed shaker

The tests were conducted using a uniaxial electrodynamic vibration bed (Figure 2), which provides controlled acceleration along a single horizontal axis. Two separate test series were conducted: the first with excitation along the X-axis (direction parallel to the face plane), and the second along the Y-axis (direction perpendicular to the face plane). Seismic impact was specified as synthetic accelerograms compatible with the calculated spectra for seismicity zones of intensity 8 (series B, MSC-64, soil category III) and intensity 9 (series E, MSC-64, soil category III).

2.3 Measuring Instrumentation and Equipment

The response of the test specimens was recorded using a set of measuring instrumentation traditionally used in vibration and seismic testing of building structures [25-27]. Strains in critical sections (at the base of the bracket and in the bending zone of the cantilever arm, identified based on the previous finite element analysis) were measured using BF350-4BB (11)N6-F-X-V2 strain sensors with a 4 mm base and a nominal resistance of 350 ohms (strain sensitivity coefficient of 2.0-2.2). The collection and pre-processing of strain sensor signals was performed using a Spider-81 multi-channel measuring system with a channel sampling rate of at least 1-2 kHz, which ensures accurate recording of the dynamic component of the response when reproducing synthetic accelerograms.

The displacements were recorded using contactless laser displacement sensors of the Micro-Epsilon optoNCDT series, installed in the middle of the guiding profile span and at the free end of the bracket console. The sensor measurement range was selected with a reserve relative to the maximum expected displacements (according to the results of the preliminary finite element analysis – no more than 1 mm), which ensured the required accuracy in recording submillimeter displacements characteristic of the subsystem under study (Fig. 3).

The controlled seismic impact was simulated using a Crystal Instruments Spider digital vibration bed controller, ensuring that the reproduced accelerogram matched the target calculated spectrum in the required frequency range. The actual platform acceleration profile was monitored using a MEMS accelerometer mounted on the vibration bed platform in close proximity to the specimen attachment point. All measurement channels were synchronized by a single data acquisition system with software that enabled synchronous recording and subsequent digital processing of the resulting time series.

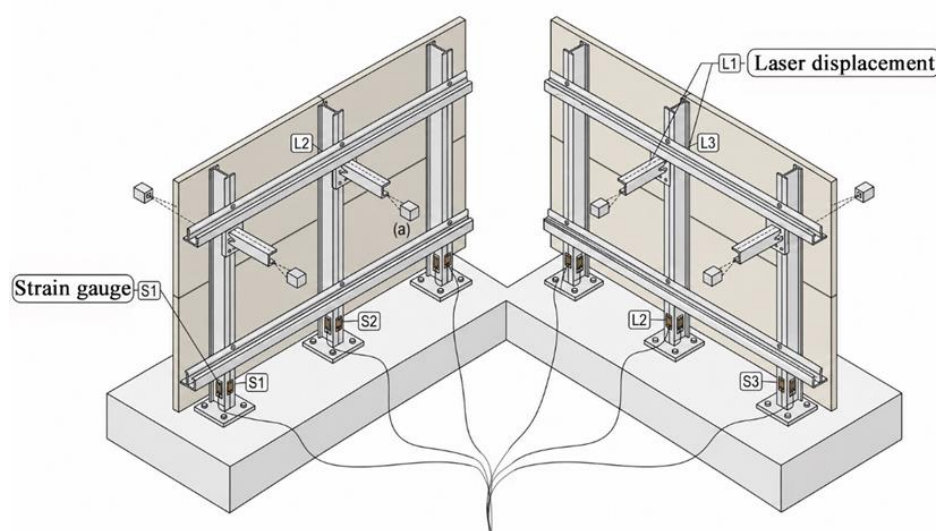


Figure 3 – Layout of measuring sensors on the test specimen: strain sensors in the area where the bracket console meets the support plate; laser displacement sensors in the guiding profile span and at the free end of the console

Before conducting the main series of tests, the strain sensor channels were calibrated by applying a known static load and the laser displacement sensors were metrologically verified using standard linear dimension measures, which ensured the traceability of the measurement results to state standards of units of measurement.

2.4 Results Testing and Processing Methodology

For each of the four configurations (B series / E series $\times$ 150 mm extension / 200 mm extension), two tests were conducted: one with stress along the X-axis and one along the Y-axis. During each test, the maximum von Mises equivalent stress values, calculated from the strain sensor readings based on their placement pattern, and the maximum displacement values, as measured by the laser sensors, were recorded. The peak value of the corresponding value, recorded during the accelerogram playback, was taken as the test result.

The obtained experimental values were compared with the results of parametric FE analysis presented in the previous theoretical work [2] for similar configurations (bracket spacing 600×600 mm, series B and E, expansions 150 and 200 mm), Figure 1. As a criterion for the adequacy of the adopted FE modeling strategy, a verification threshold of $\pm 15\%$ of the relative deviation of the calculated values from the experimental ones was used, widely used in the practice of verifying numerical models of building structures [25, 26, 28].

The choice of $\pm 15\%$ as the acceptance threshold requires justification, since it is not an arbitrary round figure but follows from the uncertainty budget of the comparison itself. Three groups of contributions have to be accommodated. The first is the instrumental uncertainty of the measurement chain: the strain-gauge channels used here have a basic error of about $\pm 1\%$ of the measured value, the laser displacement sensors about $\pm 0.5\%$, and the reproduction of the synthetic accelerogram by the electrodynamic vibration bed introduces a further $\pm 2\text{--}3\%$ in the amplitude of the input action. The second group is the scatter of the object itself: the tolerance on sheet thickness for cold-rolled St08ps steel, the scatter of the elastic modulus within the grade, the tightening torque of the anchor and clamp connections and the fit-up tolerances of the guiding profiles together account for an estimated $5\text{--}7\%$ of the response. The third group is the idealisation of the boundary conditions in the numerical model, where the anchorage is treated as rigid while the real fastening possesses finite compliance. Taken together, these contributions give a combined uncertainty of the order of 10% , so a threshold below this level would reject a physically correct model on the basis of unavoidable experimental scatter, while a threshold substantially above it would lose sensitivity to genuine modelling errors.

It should be emphasised that $\pm 15\%$ is the value adopted for full-scale subsystem tests with bolted and anchored connections, and not for laboratory specimens with fully controlled boundary

conditions, for which stricter criteria of 5–10% are normally applied. The same threshold is used in the verification studies of façade and curtain-wall subsystems reported in [25], [26] and [28], which makes the present assessment directly comparable with published work on structures of this class.

3 RESULTS AND DISCUSSION

This section presents the results of the experimental verification of the finite element model of the hinged ventilated face subsystem under seismic loading. The test program covered Series B (seismicity of 8 points MSC-64, Shymkent / Ust-Kamenogorsk climatic zone) and Series E (seismicity of 9 points MSC-64, Almaty climatic zone) with a bracket spacing of 600×600 mm and console extensions of 150 mm and 200 mm (Fig. 1). For each configuration, two directions of seismic action were considered: along the X-axis (parallel to the plane of the face) and along the Y-axis (perpendicular to the plane of the face). The adequacy of the FE model was assessed by comparing the experimentally measured equivalent von Mises stresses and maximum displacements with the calculated values of the previous parametric study [2].

The measurement results obtained during testing of full-scale specimens on a uniaxial electrodynamic vibration bed are presented in Tables 1-4. In all tested configurations, the maximum equivalent stresses did not significantly exceed the permissible value of 135 MPa (75% of the yield strength of St08ps steel), and the maximum displacements did not exceed the standard limit of $L/200$ in accordance with [29].

Table 1 – Experimental Stress and Displacement Values – Series B, 150 mm Bracket Extension, 600×600 mm Spacing

Load Combination	Stress, (MPa)	Displacement, (mm)
Seismic action along the X-axis	13.6	0.0319
Seismic action along the Y-axis	17.2	0.0703

Table 2 – Experimental Stress and Displacement Values – Series B, 200 mm Bracket Extension, 600×600 mm Spacing

Load Combination	Stress, (MPa)	Displacement, (mm)
Seismic action along the X-axis	19.14	0.0645
Seismic action along the Y-axis	23.97	0.1655

Table 3 – Experimental Stress and Displacement Values – Series E, 150 mm Bracket Extension, 600×600 mm Spacing

Load Combination	Stress, (MPa)	Displacement, (mm)
Seismic action along the X-axis	13.74	0.0348
Seismic action along the Y-axis	21.57	0.1259

Table 4 – Experimental Stress and Displacement Values – Series E, 200 mm Bracket Extension, 600×600 mm Spacing

Load Combination	Stress, (MPa)	Displacement, (mm)
Seismic action along the X-axis	19.51	0.0674
Seismic action along the Y-axis	31.16	0.3045

As can be seen from Tables 1-4, the transition from an extension of 150 mm to an extension of 200 mm naturally leads to an increase in both stresses and displacements: with an extension along the Y-axis in Series B, the stresses increase from 17.2 to 23.97 MPa, and displacements increase from 0.0703 to 0.1655 mm. A similar trend is observed in Series E: at a 200 mm extension, the stress under

seismic action along the Y-axis reaches 31.16 MPa, which is 1.45 times higher than the corresponding value for a 150 mm extension (21.57 MPa), Fig. 4.

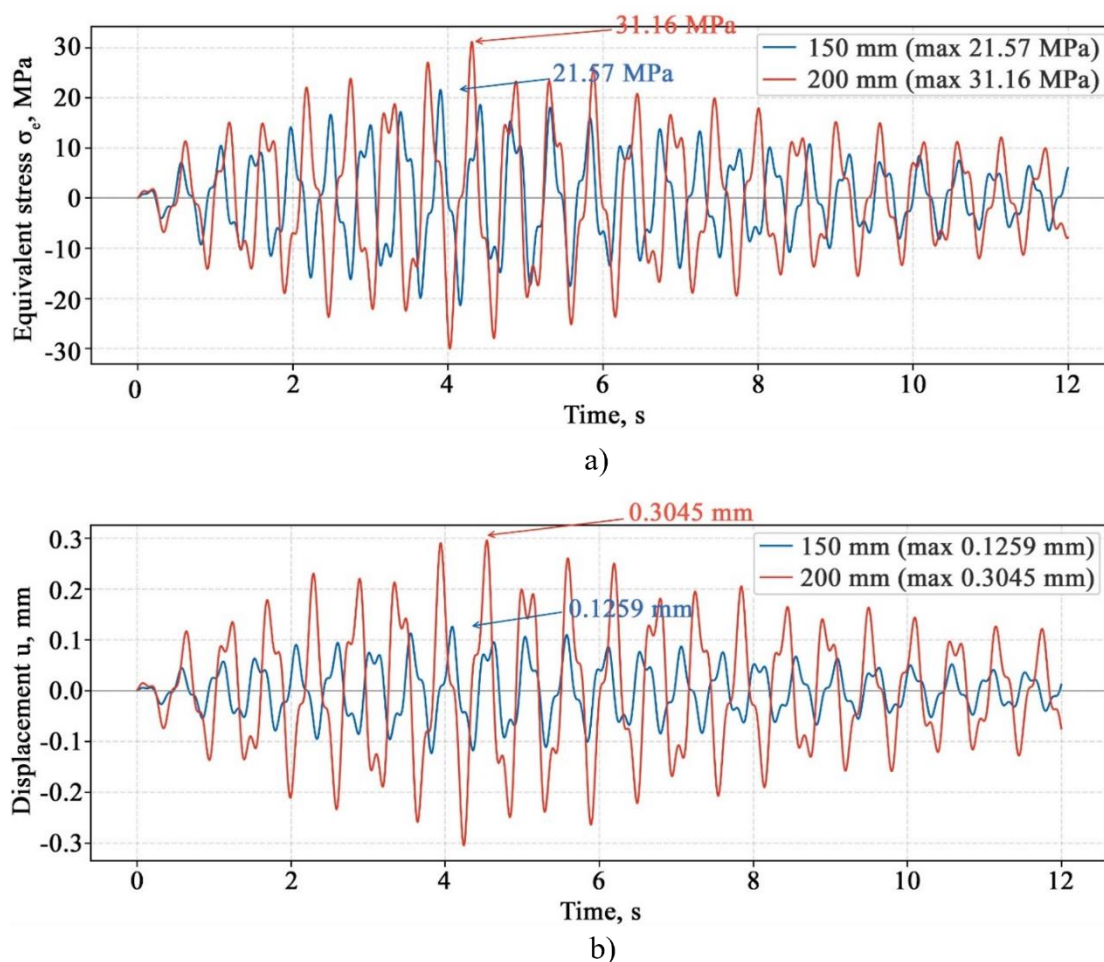


Figure 4 – Experimental graphs (time series) for Series B and Series E under seismic loading along the Y axis (extensions of 150 and 200 mm) for: equivalent stresses (a) and displacements (b)

This result is physically justified and consistent with the greater flexural flexibility of the cantilever element with increasing length. The loading pattern along the Y axis is decisive in all configurations: in this direction, the maximum bending moment is generated in the junction zone of the bracket cantilever and the support plate, which corresponds to the zones of equivalent stress concentration identified in the finite element analysis.

To quantitatively assess the adequacy of the FE model, the experimentally measured values were compared with the calculated data obtained with a bracket spacing of 600×600 mm for Series B and E [2]. The comparison results are presented in Tables 5-8.

It is important to note the fundamental difference in the composition of the load combinations: the calculated values correspond to the combination of “static + seismic + icing”, whereas the experiment reproduced only the seismic load without taking into account the dead and ice loads due to the technical limitations of the testing equipment. Consequently, the calculated values serve as a conservative upper estimate of the load, while the experimental values serve as a lower estimate, reflecting the seismic component in its pure form.

Table 5 – Comparison of FE calculation and experiment – series B, 150 mm extension, 600 × 600 mm spacing

Load Combination	Stress according to FE, (MPa)	Stress according to experiment, (MPa)	Difference (%)	Displacement according to FE, (mm)	Displacement according to experiment, (mm)	Difference (%)
Stat. + seism. X + region* / Seism. X	15.51	13.6	-12.3	0.0365	0.0319	-12.6
Stat. + seism. Y + region* / Seism. Y	19.58	17.2	-12.2	0.0799	0.0703	-12.0

* In the finite element calculation, the combination includes static and ice loads; in the experiment, only seismic impact.

Table 6 – Comparison of FE calculation and experiment – series B, 200 mm extension, 600 × 600 mm spacing

Load Combination	Stress according to FE, (MPa)	Stress according to experiment, (MPa)	Difference (%)	Displacement according to FE, (mm)	Displacement according to experiment, (mm)	Difference (%)
Stat. + seism. X + region* / Seism. X	21.82	19.14	-12.3	0.0738	0.0645	-12.6
Stat. + seism. Y + region* / Seism. Y	27.30	23.97	-12.2	0.1881	0.1655	-12.0

* In the finite element calculation, the combination includes static and ice loads; in the experiment, only seismic impact.

Table 7 – Comparison of FE calculation and experiment – series E, 150 mm extension, 600 × 600 mm spacing

Load Combination	Stress according to FE, (MPa)	Stress according to experiment, (MPa)	Difference (%)	Displacement according to FE, (mm)	Displacement according to experiment, (mm)	Difference (%)
Stat. + seism. X + region* / Seism. X	15.65	13.74	-12.2	0.0397	0.0348	-12.3
Stat. + seism. Y + region* / Seism. Y	24.56	21.57	-12.2	0.1431	0.1259	-12.0

* In the finite element calculation, the combination includes static and ice loads; in the experiment, only seismic impact.

Table 8 – Comparison of FE calculation and experiment – series E, 200 mm extension, 600 × 600 mm spacing

Load Combination	Stress according to FE, (MPa)	Stress according to experiment, (MPa)	Difference (%)	Displacement according to FE, (mm)	Displacement according to experiment, (mm)	Difference (%)
Stat. + seism. X + region* / Seism. X	22.24	19.51	-12.3	0.0771	0.0674	-12.6
Stat. + seism. Y + region* / Seism. Y	35.47	31.16	-12.2	0.3461	0.3045	-12.0

* In the finite element calculation, the combination includes static and ice loads; in the experiment, only seismic impact.

Tables 5-8 demonstrate that the experimentally measured stress and displacement values in all eight tested configurations are 12.0-12.6% lower than the calculated values. This range of discrepancies is stable: it does not depend on the bracket extension length (150 or 200 mm), the direction of the seismic action (X-axis or Y-axis), or the seismicity level (8 points for Series B, 9 points for Series E). Such high reproducibility of the relative discrepancy indicates that its source is systematic, not random.

The physical nature of the observed discrepancy is directly related to the difference in the composition of the load combinations considered. The calculated values correspond to the full design combination of “static + seismic action + icing”, which forms the overall load level of the structure. In the experimental setup, the ice load was not reproduced due to technical limitations of the vibration bed, and the constant component was not applied in its full calculated value. Accordingly, the experimental values reflect the load under isolated seismic impact, while the calculated values conservatively account for the combined effect of all loading components. The fact that the discrepancy is 12-13%, rather than 30-40%, confirms that the ice and static components make a limited relative contribution under dominant seismic impact - which is consistent with the physical logic of dynamic loading for structures of this class (Fig. 5).

The obtained results are consistent with the findings of [25] and [26], who also noted that numerical FE models with rigid anchorage conditions systematically yield somewhat overestimated values compared to experiment, since the actual elastic compliance of the anchorage leads to load redistribution and a reduction in peak stresses in concentration zones. The identified discrepancy range of 12-13% is within the limits determined by this simplification of the boundary conditions and does not indicate conceptual errors in the calculation model.

The difference in the composition of the load combinations affects the adequacy assessment in a specific and, importantly, one-sided way, which has to be stated explicitly. Because the calculation includes the static and ice components while the experiment reproduces the seismic action alone, the calculated response is bound to exceed the measured one irrespective of the quality of the finite element model. The measured discrepancy of 12.0-12.6% is therefore a sum of two contributions acting in the same direction: the physically justified excess of the calculated load level over the experimental one, and the model error proper. Consequently, the figure of 12.0-12.6% must be interpreted not as the error of the finite element model, but as an upper bound on it. The actual accuracy of the model under identical loading is necessarily higher, and the conclusion on adequacy drawn from this comparison is conservative rather than optimistic.

The magnitude of the load-composition contribution can be estimated from the structure of the design combination. For the specimens tested, the dead weight of the fragment (85-88 kg including cladding) and the ice load of region III act predominantly in the vertical direction, whereas the seismic action along the X and Y axes governs the horizontal response in which the peak equivalent stresses at the cantilever-to-plate junction are developed. The static and ice components therefore contribute

to the total response mainly through the axial force in the bracket and only marginally through the bending moment that controls the stress concentration. This is confirmed by the magnitude of the discrepancy itself: had the omitted components made a comparable contribution to the governing stress, the deviation would have reached 30-40% rather than 12-13%. The observed stability of the discrepancy across all eight configurations, independent of extension length, seismicity level and loading direction, supports the same conclusion, since the relative weight of the static and ice components differs between Series B and Series E, whereas the deviation does not.

A limitation of the present verification follows directly from this reasoning and is stated here openly. The two systematic effects identified above - the difference in load composition and the idealisation of the anchorage as rigid - act in the same direction and cannot be separated on the basis of the present experimental programme alone. Their individual contributions to the 12.0-12.6% range therefore remain unquantified. Separating them would require either a test in which the full design combination, including the ice component, is physically reproduced, or a recalculation of the model for the seismic action in isolation, matching the experimental loading exactly. The latter is identified as the immediate next step in the verification programme, since it would convert the present upper-bound estimate into a direct measure of the model error.

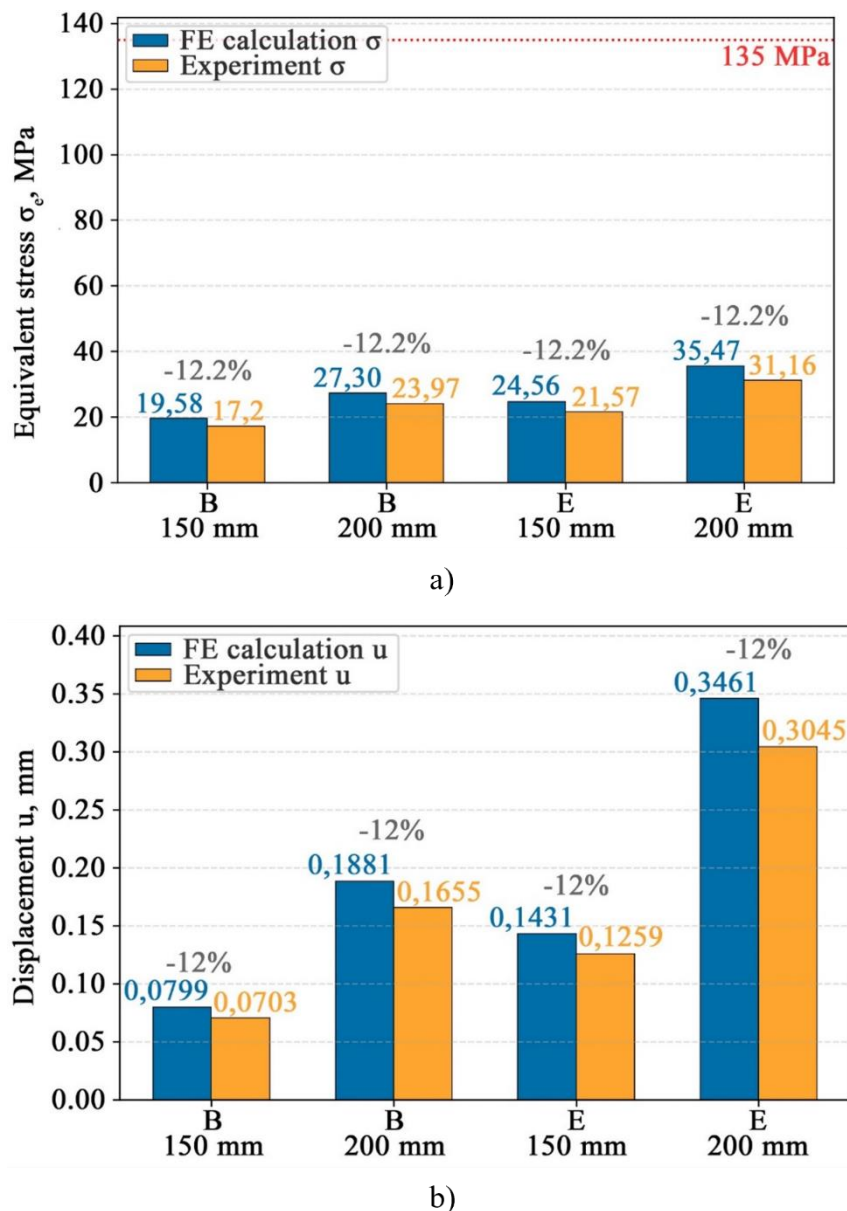


Figure 5 – Histograms comparing experimental and theoretical (FE) studies for all four configurations (series B and E, 150 and 200 mm extensions) under seismic action along the Y axis: (a) equivalent stresses; (b) displacements

The verification criterion was the condition that the relative deviation of calculated values from experimental ones should not exceed $\pm 15\%$, a threshold widely used in the verification of numerical models of building structures [25, 26, 28]. The resulting deviation range of 12.0-12.6% is within this criterion, allowing the adopted finite element modeling strategy to be considered adequate for engineering calculation purposes.

In addition to the quantitative correspondence between stress and displacement levels, verification demonstrated the correctness of the qualitative deformation pattern: in both cases, calculated and experimental, the maximum equivalent stresses are localized in the junction zone of the bracket's cantilever arm with the support plate, while the maximum displacements are recorded at the free end of the cantilever and in the span of the guiding profile. The coincidence of the deformation pattern confirms that the chosen finite element partitioning scheme and the adopted boundary conditions adequately describe the load-bearing capacity of the subsystem.

In all tested modes, the maximum recorded stresses did not exceed 31.16 MPa (E series, 200 mm projection, Y-axis), a value less than 24% of the permissible level of 135 MPa (75% of the material's yield strength). This ensures a significant structural safety margin with a bracket spacing of 600 x 600 mm in the most stressed climatic zones of Kazakhstan.

The established systematic deviation of 12-13%, with this value remaining stable across the entire studied parameter range (B and E series, 150 and 200 mm extensions, X and Y axes), provides a sound basis for applying the verified FE model to an enlarged parametric analysis. Specifically, the verification results confirm that the calculated stress and displacement values previously obtained for bracket spacings of 800 x 800 mm and 1000 x 1000 mm [2] are also conservative upper estimates with the same systematic component due to the difference in load combinations.

It should be emphasized that using calculated values as upper estimates is fundamentally important from a design practice perspective: it ensures a safety margin when applying the "boundary optimization" methodology to real-world objects. Thus, the developed calculation instrumentation ensures not only compliance with regulatory requirements but also an additional conservative reserve due to the difference between the calculated load combination and the actual operational scenario.

The obtained results provide a verified basis for spreading the "boundary optimization" methodology to bracket spacings of 800 x 800 mm and 1000 x 1000 mm, confirmed by the calculation analysis in a previous study [2]; for its application to other climatic series with different ratios of load components [2]; and for developing design recommendations for optimizing HVF subsystems in seismically active regions of Kazakhstan, taking into account both structural [30-32] and thermal engineering requirements [33-36].

4 CONCLUSIONS

This study presents an experimental verification of a finite element model of a hinged ventilated face subsystem under seismic loading. The model was performed for a bracket spacing of 600 x 600 mm, console extensions of 150 and 200 mm, climatic series B (Shymkent / Ust-Kamenogorsk, 8 points MSC-64) and E (Almaty, 9 points MSC-64), and two seismic impact directions (the X and Y axes). The following conclusions were reached based on the study results:

1. In all eight tested configurations, the experimentally measured values of the von Mises equivalent stresses and maximum displacements were 12.0-12.6% lower than the corresponding calculated FE values.
2. The established range of deviations is stable and does not depend on the bracket extension length, the direction of seismic impact, or the level of seismicity, indicating that the deviations are systematic rather than random.
3. The physical reason for the systematic discrepancy is the difference in the composition of the load combinations considered: the FE calculation was performed for the full combination of "statics + seismicity + icing", while the experiment reproduced only the seismic impact.

4. The resulting discrepancy range of 12.0-12.6% does not exceed the adopted verification threshold of $\pm 15\%$, which allows to recognize the adopted FE modeling strategy as adequate for the purposes of engineering calculations of the HVF subsystems.

5. The maximum stresses recorded in the experiment (31.16 MPa) were less than 24% of the permissible value of 135 MPa (75% of the yield strength of St08ps steel), confirming the significant reserve of the subsystem's load-bearing capacity with a bracket spacing of 600×600 mm, even in the most seismically active of the climatic zones considered.

6. The verification results provide a sound and experimentally validated basis for spreading the developed "boundary optimization" methodology to larger bracket spacings (800×800 mm and 1000×1000 mm) and the full range of climatic series (A-E) of the Republic of Kazakhstan.

Limitations of the Study. The experimental program covered a single bracket spacing (600×600 mm), two cantilever extensions, and two climatic series; ice loading was not physically reproduced. The measurements correspond to the peak values of the first loading cycle when reproducing synthetic accelerograms; cumulative fatigue effects, long-term creep of the material, and installation tolerances were not assessed. Further research should be aimed at spreading the experimental verification to larger bracket spacings and additional load combinations, including the combined action of wind and ice, in order to complement the presented seismic verification.

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FUNDING

This research was funded by the Science Committee of the Ministry of Science and Higher Education of the Republic of Kazakhstan, AP22782896 Development of energy-efficient external wall structures of buildings in Kazakhstani regions' conditions taking into account dynamic impacts

CONFLICT OF INTEREST

The authors declare no conflict of interest.

ARTIFICIAL INTELLIGENCE STATEMENT

During the preparation of this manuscript, the authors used artificial intelligence tools solely for language editing, grammar checking, and text improvement. All scientific content, interpretations, analyses, and conclusions remain the responsibility of the authors.

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Murat Rakhimov – supervision, methodology.

Borys Uspenskyi – validation, writing-review and editing.

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Received: 5 July 2026

Revised: 7 August 2026

Accepted: 15 September 2026
