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STATIC EFFECTS OF MODULAR STRUCTURES MADE OF CONTAINERS

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ABSTRACT

When designing and static assessing of modular buildings, it is necessary to take into account some specificities and differences related to the static effect of the load carrying structure made of interconnected steel containers. The objective of container module manufacturers is to achieve a broad application of their products in building construction. The load carrying structure of containers used in building construction was created by modification of original containers used mainly in rail and ship transport. The necessary condition for the application of the products in building construction is to ensure the mechanical resistance and the stability of the supporting structure in accordance with requirements of the applicable standards. This paper therefore points to selected static problems associated with the use of containers when designing multi-storey modular buildings.

Keywords: modular building, load bearing structure, mechanical resistance, stability, container.

1. INTRODUCTION

The modular container system was developed for fast and time-consuming construction. The modular construction is based on three basic assumptions: prefabrication of components in a production hall, mobility of the components for simple transport to the site and variability of the component to create the necessary composition of the building. Modules can be combined together to create different assemblies according to the purpose and the location of the building. The required layout and aesthetic appearance of the structure are achieved by variants of filling walls between the main load carrying elements of the individual containers or by complete omission of them. An example of a multi-storey modular building composed of containers is shown in Figure-1. For more detailed information about the possibilities of application of modular container structures, see [1], [2].


Figure-1. Multi-storey modular building from containers (a civic amenities facility at the airport Tegel [1]).

The main load carrying structure of the steel container is designed as a space frame, consisting of corner columns and longitudinal and transverse girders, see Figure-2 (on the left). The floor and the ceiling are laid on transverse beams. The entire structure of the container is designed as welded. All beam elements are made of cold-formed cross sections. The knee of rigid frame consists of a massive corner cube, to which columns and girders are welded. The ceiling and wall cladding are designed from molded sheet, fastened to a wall grid by rivets and self-tapping screws. The containers are interconnected at the corners (vertically and horizontally) by mechanical anchors, see Figure-2 (on the right).

The container system statically acts as a spatial structure, consisting of frames of individual containers. The vertical load is transferred from the floor or roof beams to the knees of frames through the longitudinal girders. The transfer of load effects between adjacent containers takes place only at the corners, the joint statically acts as an articulated joint. Compression forces in the joint are transmitted by contact, tension forces and shear forces are transmitted by mechanical coupling means.

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Figure-2. Load carrying elements (on the left) and mechanical coupling of containers [1] (on the right).

The load carrying structure of modular buildings is not too complicated, however in the static assessment of structures, it is necessary to pay special attention to some specific problems arising from the non-traditional structural system. The design of multi-storey modular buildings can be influenced mainly by the following limiting factors:

a) ensuring a sufficient horizontal stiffness of the structure,
b) static assessment of knees of rigid frame.

This paper deals mainly with the issue of ensuring a sufficient horizontal stiffness of the structure, see chapter 2. The issue of static assessment of knees of rigid frame is only briefly indicated in chapter 3 at the end of the paper.

2. HORIZONTAL STIFFNESS OF THE STRUCTURE

The horizontal stiffness of the structure is primarily ensured by rigid joints between the columns and the girders of the individual containers. However, the main load carrying elements of the container are relatively subtle and the horizontal stiffness of the open frame, i.e the frame without cladding of molded sheet, is significantly affected by this fact. The stiffness of the adjoining containers is negligible, and therefore, an articulated joint is assumed in a static beam model (vertical connection is usually modeled using a short, rigid beam with an articulated joint at the end). With respect to these facts, it is clear that meeting the condition of Serviceability limit state (SLS) related to the maximum permissible value of horizontal object deformation may be the decisive criterion limiting the use of containers for multi-storey objects.

With respect to the required layout of the building, it is usually possible to have some walls with continuous cladding of molded sheet reinforced with a wall grid (type solution of IMECON Containers, a.s.). For example, the gable walls of the object are typically solved this way. Due to the low stiffness of the container frame in the horizontal direction, it is therefore possible to use the cladding as an additional stiffening diaphragm.

In order to determine the real horizontal stiffness of the cladding wall of the container, a program of experimental tests for a standard wall system of the container was carried out in cooperation with the container manufacturer IMECON Containers, a.s. (for the comparison, the stiffness of the separate steel frame of the container was verified). An extensive experimental program of specimens of real dimension took place at the premises of the Ostrava branch of TZÚS Praha, s.p. The tested container wall was fastened to a pre-set gripping cage, and after the deformation and strain gauges were installed, a horizontal load was applied in the upper corner. The wall was loaded until it was completely damaged, the wall was completely unloaded and then reloaded several times in the area of elastic deformations. The concepts and basic dimensions of the physical test and photographs of the selected wall after completion of the measurement are shown in Figure-3.

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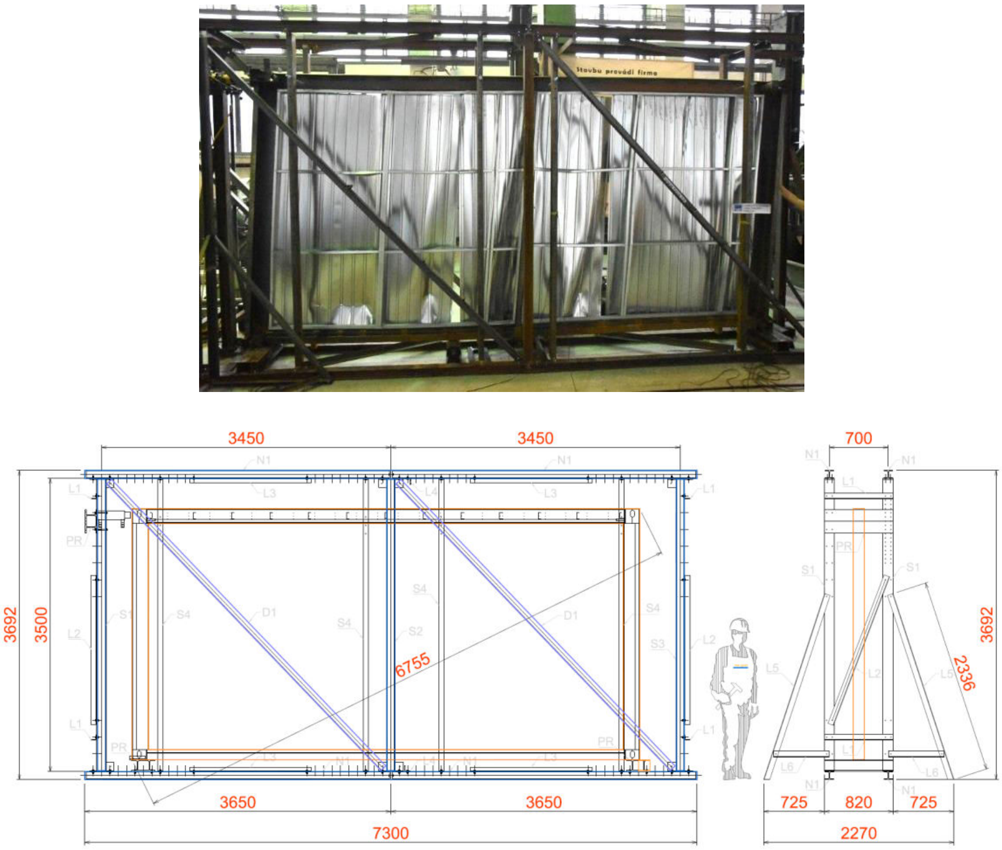

Figure-3. Concepts and basic dimensions of the physical test (below) and the damaged wall after completion of the measurement (above)

During the step loading of the cladded walls, a local buckling of the sheet first appeared; then with the increasing deformation, a failure of the self-tapping screws and rivets occurred; and the final damage was caused by the crack in the weld of the plastic-deformed knee of rigid frame, see Figure-4.

Figure-4. Crack in the weld of the knee of rigid frame (selected examples).

The results of the experimental measurement show that the wall formed by profiled sheet (TR9/123-0,55 in the experiment) contributes significantly to the increase of the horizontal stiffness of the structure. Comparison of the experimentally determined data for the non-cladded and cladded wall of the container is shown in Figure-5.

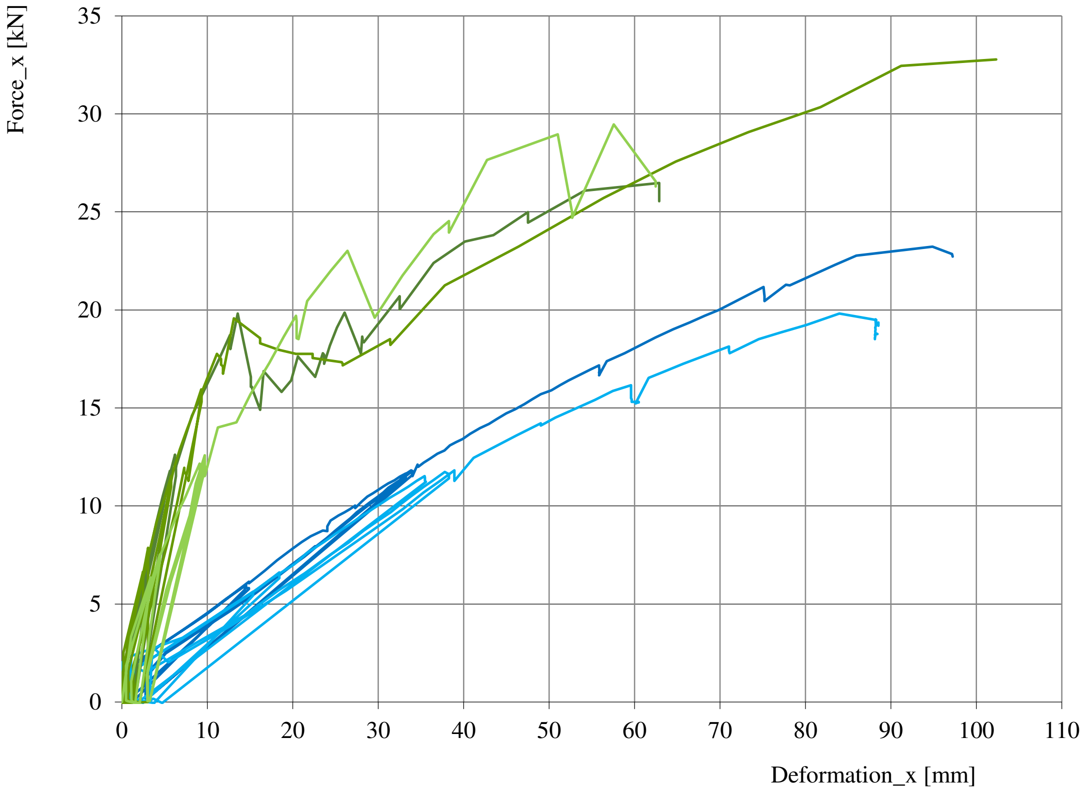

Figure-5. Comparison of the experimentally determined data of the horizontal stiffness (green records - cladded wall, blue records - non-cladded wall).

Experimentally found data can be used to verify numerical models. With the use of available static software, it is possible to create a shell model of a wall cladded by a profiled sheet with a reinforcing wall grid. The shell model shown in Figure-6 (above) was created in SCIA Engineer and the initial stiffness of the numerical and experimental model was equal. Using numerical models, it is possible with sufficient accuracy to determine contribution of the shear stiffness to the overall horizontal stiffness of the container wall. The determined value of the additional horizontal stiffness can then be implemented into the overall beam model of the structure, for example, using fictitious rigid diagonals ended by the axial flexible support, see Figure-6 (in the middle). The stiffness of the flexible support can be tuned by comparing the deformations of the shell and the beam model, which must be identical for both models.

The substitution of wall stiffness by using fictitious diagonal elements with axial flexible support is correct only if the horizontal load applied to the wall of the container does not exceed the value corresponding to the elastic effect of the wall. As the experimental loading first resulted in a local buckling of the profiled sheet, the horizontal load limit can be derived from the results of the nonlinear stability analysis performed on the above shell model. For the container wall shown in Figure-6 loaded at the top by a horizontal force F = 5 kN, the non-linear stability calculation determined the critical factor a_{cr} = 3.05 for the first mode of buckling, the deflection shape is shown in Figure-6. When verifying the suitability of the beam model, it is necessary to verify that the horizontal force acting on the cladding wall (horizontal force can be determined from the reactions) does not exceed the value guaranteeing the elastic effect of the cladding wall $F_{max} = a_{cr} \cdot F = 3.05 \cdot 5 = 15.3$ kN (in the static calculations of the various container assemblies made by the authors of this paper, this criterion has always been met).

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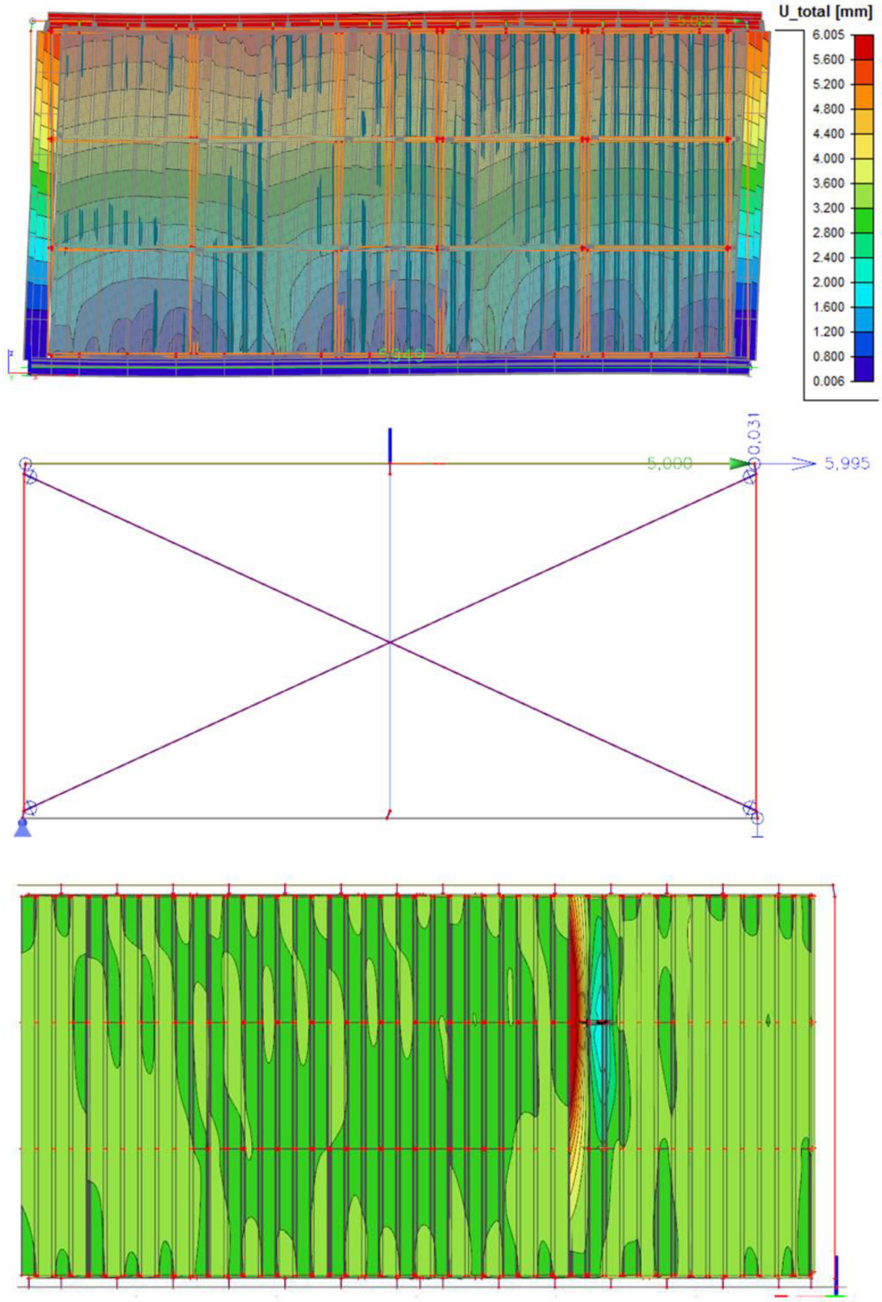

Figure-6. Numerical model of the cladded wall of the container (above - numerical model using shell elements, in the middle - beam wall model with fictitious rigid diagonals ended by the axial flexible support, below - loss of stability cause by horizontal force F = 5 kN for the first mode of buckling: acr = 3.05)

3. STATIC ASSESSMENT OF KNEES OF RIGID FRAME

The results of the static analysis carried out using the spatial beam models of the container assemblies show that the main supporting elements of the containers (columns and girders) are most used statically in the area of the knees (the elements are designed to almost 100% of their load carrying capacity according to the applicable standards). However, the structural design of the welded knees is so complicated that it cannot be modeled with sufficient precision using beam models.

Based on the practical experience of the authors of this paper, a conservative recommendation for carrying out a detailed analysis of the structure in the knee area is provided. If one of the attached members in the knee (using a beam model) is statically designed to more than 80% load carrying capacity determined in accordance with the applicable standards. The authors of the article have good experience for these cases using numerical models combining beam and shell elements - the whole structure is considered as a beam model (using the original beam model) except the knee of rigid frame and the appropriate length of the attached beams, see Figure-7. Geometrically

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and Materially Nonlinear Analysis with Imperfections (GMNIA) [2] is used for the calculation of the load effects (for example using SCIA Engineer).

IMPERFECTIONS INTRODUCED INTO THE NUMERICAL MODEL:

a) Local imperfection (beam): deformation of a beam (l / 200);
b) Global imperfection (beam): from the load case, or buckling length from the stability analysis;
c) Local imperfection (shell): shear buckling length from the stability analysis.

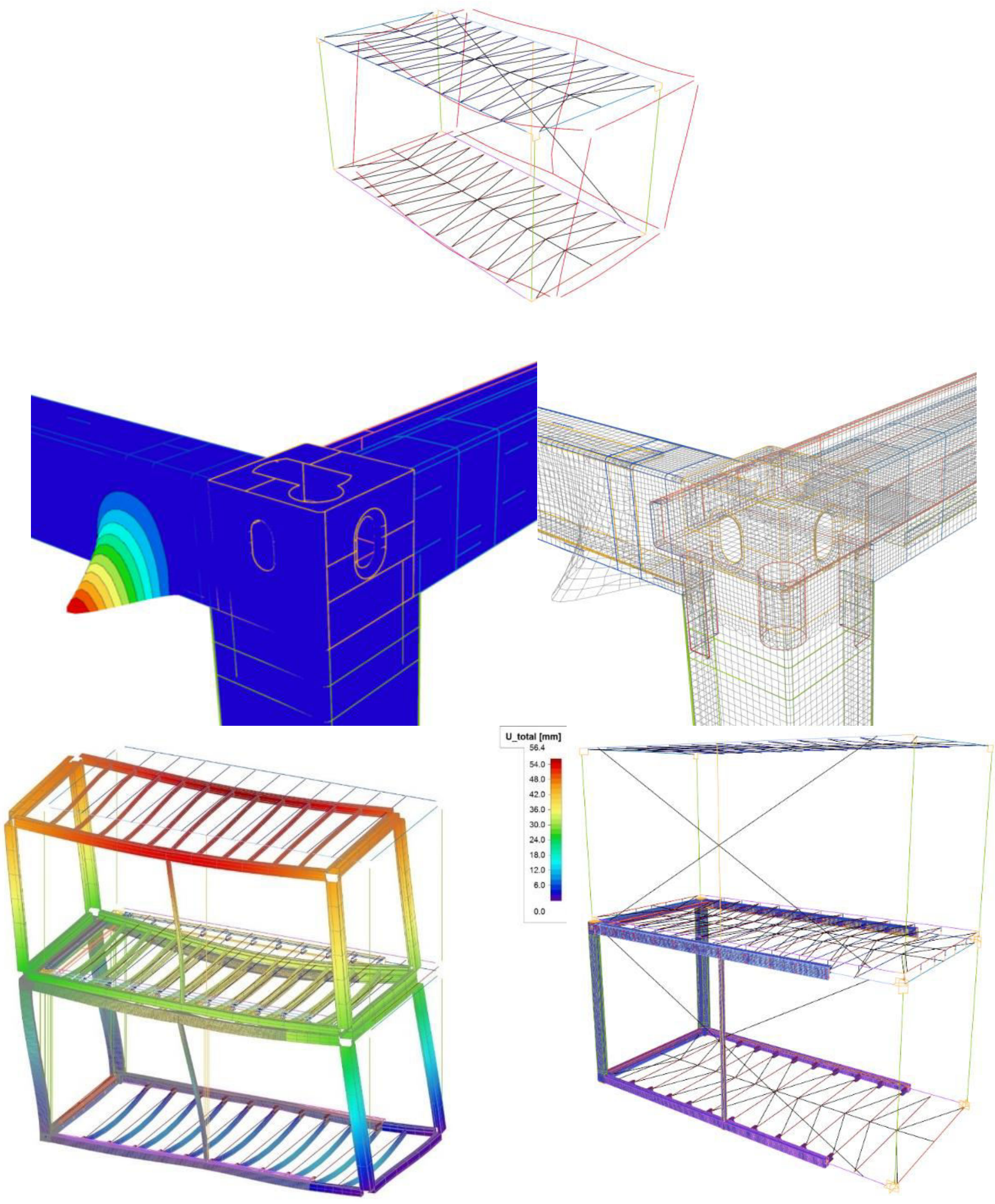

Figure-7. Numerical model combining beam and shell finite elements.

For the Ultimate limit state (ULS) assessment of parts of the structure modeled with shell elements, the criteria for allowable total and plastic strain given in the standard EN 1993-1-5 can be used. The first criterion determines the recommended value for the total principal strain on the wall surface (ie, including the momentum strain of the walls) by $\varepsilon \leq 0.05$ (5 %). Another criterion recommends limits of the permanent (plastic) component of the strain on the median plane (membrane deformation) by $\varepsilon_{m,pl} \leq 0.002$ (0.2 %).

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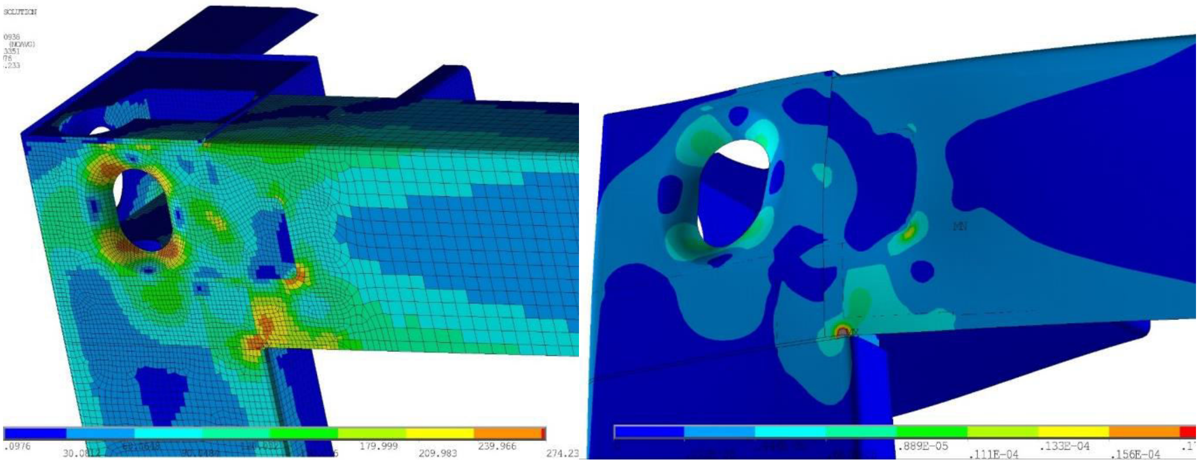

Figure-8. Knee of rigid frame - course of stress (above) and total strain (below) n the outer surface of the shell model determined by (GMNIA)

The material properties of steel are implemented into the structural analysis in accordance with Annex C of EN 1993-1-5. The use of the stress-strain diagram for an ideal plastic behavior is not very appropriate due to the difficult convergence of the geometrically and materially nonlinear model. For structural analysis using conventional commercial software, it is therefore advisable to use a stress-strain diagram with strain hardening. Authors of the paper commonly use the stress-strain diagram with strain hardening (E / 100).

The simplicity of numerical models is verified by comparing the outputs of numerical analysis to the results of the tests. To model material behavior in numerical models, it is advisable to use the real stress-strain diagram corresponding to the tensile test results of steel. Therefore, specimens for the tensile test according to EN ISO 6892-1 were taken from the structure designed for the physical testing. Tensile testing was carried out in an accredited laboratory of Arcelor Mittal Ostrava. 10 specimens were tested using the Instron 4210 laboratory press. The results of the tensile test are shown in Figure-9. The average value of the yield strength of the steel is 270 MPa. The structures of the containers are made of S235JR steel grade. The difference of the yield strength in relation to the nominal value of f_y = 235 MPa given in EN 1993-1-1 is common for S235 steel grade.

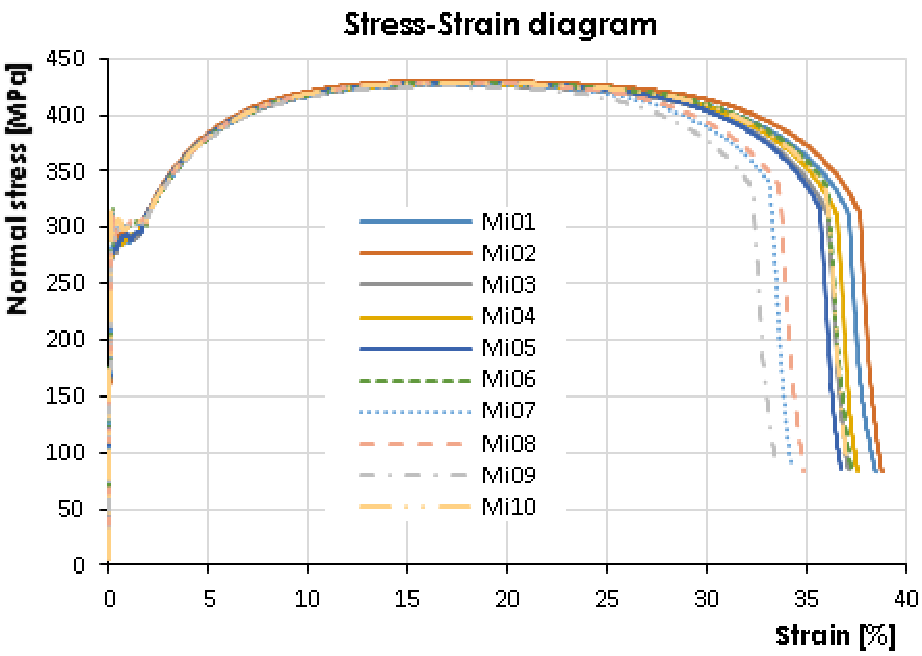

Figure-9. Stress-strain diagrams of 10 steel samples.

The comparison between the real behaviour of the non-cladded container wall received from the physical testing, and the results of the numerical analysis is shown in Figure-10 (curves corresponding to the physical models

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are limited due to the measurement range of extensometers). Based on the curves shown in Figure-10, there is a very good agreement between the results of the experimental testing and the results of the nonlinear analysis with the implementation of the real stress-strain diagram. Thus, it is possible to state that nonlinear numerical analysis using shell finite elements can be used to accurately model the real behaviour of the load carrying structure of the modular buildings composed of containers. Figure 9 also shows the differences resulting from the use of various stress-strain diagrams. As expected, the most conservative results are achieved using the stress-strain diagram for an ideal plastic behavior. The red arrow shows the state of the structure corresponding to the principal strain on the surface $\varepsilon = 0.05$.

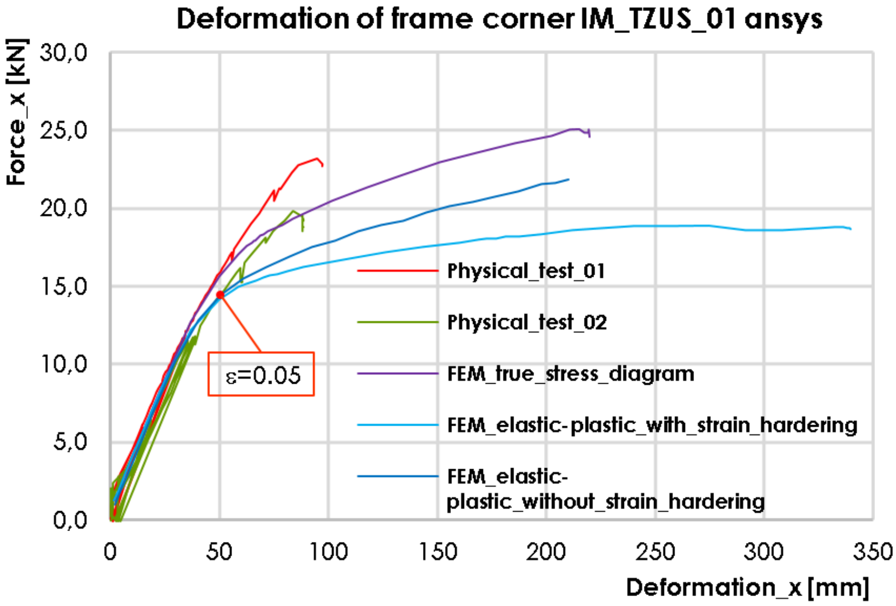

Figure-10. Comparison of results of physical tests with results of numerical analysis.

4. MECHANICAL COUPLINGS (VERTICAL AND HORIZONTAL)

Containers placed side by side or on top of each other are attached together in knees of rigid frame by mechanical couplings. The mechanical couplings are fitted in the suspension lugs and the prestressing force is applied by tightening the bolt. There are two types of the mechanical couplings. The first type is the horizontal coupling (see Figure-11) and the other type is the vertical coupling (see Figure-12).

Behavior of the joint can be simply expected as articulated with the transmission of horizontal forces. Based on the request made by IMECON, physical and numerical tests of mechanical couplings will be carried out. The tests will focus on the translational and rotational stiffness of the joint. The real stiffness can be implemented to computationally unpretentious beam models.

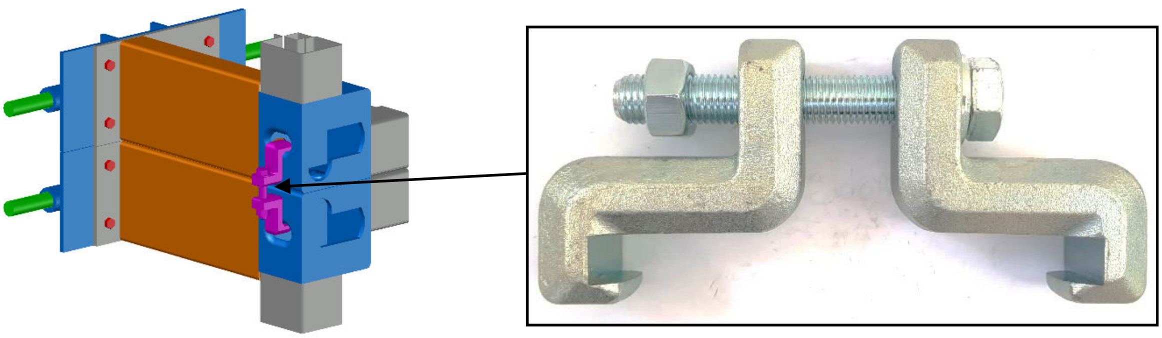

Figure-11. Horizontal coupling.

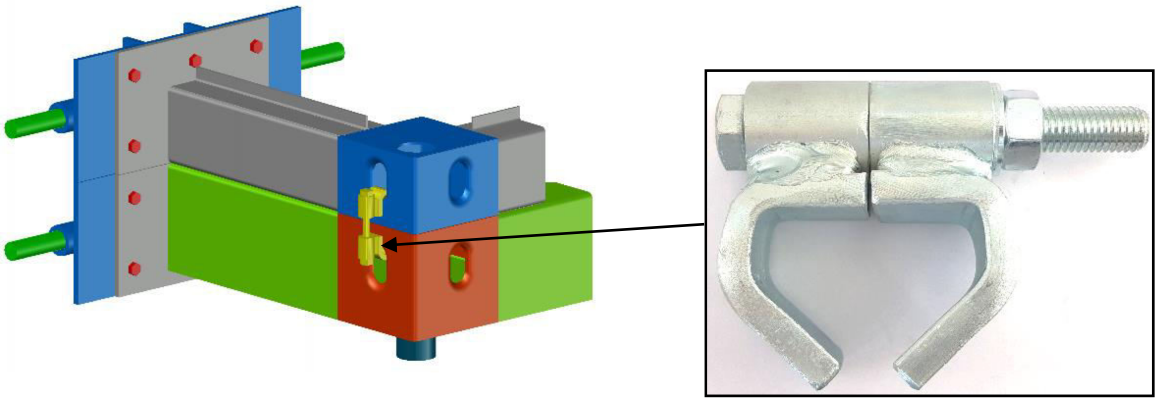

Figure-12. Vertical coupling.

5. CONCLUSIONS

The main objective of this paper was to share practical experience with static design of modular container buildings. The static assessment of various multi-storey modular systems developed by IMECON Containers, a.s. has to pay attention to the issue of ensuring and verifying the sufficient horizontal stiffness of the objects and the assessment of the Ultimate limit state in the knee of rigid frame of the container assembly. Commonly available static programs can be used for modeling the structure, both for the design of the beam model of the whole structure and for more complex numerical analysis using shell finite elements and how to create mock the whole structure, as well as for complex numerical analysis using shell finite elements. Container modular assemblies developed by IMECON Containers, a.s., certified using detailed numerical and experimental modeling, meet the relevant requirements for reliable behavior of the load carrying structure.

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REFERENCES

[1] https://www.imecon.cz/

[2] Křivý V., Kubzová M. 2017. Static analysis and numerical modeling of steel cylindrical tanks weakened by corrosion. Modelování v mechanice. pp. 31-32, ISBN 978-80-248-4010-9

[3] KALKIN Adam: 2008. Quik Build: Adam Kalkin´s ABC of Container Architecture, London.

[4] ČSN EN 1993-1-5 Eurocode 3: Design of steel structures - Part 1-5: Plated structural elements, Prague, ČNI 2006.

[5] ČSN EN 1993-1-1 Eurokód 3: Design of steel structures - Part 1-1: General rules and rules for buildings, Prague, ČNI 2007.

[6] ČSN EN 1993-1-8 Eurokód 3: Design of steel structures: Design of joints, Praha, ČNI, 2006.

[7] Virdi KS *et al*. 1999. Numerical simulation of semi-rigid connections by the finite element method. Report of Working Group 6, Numerical Simulation COST C1, Brussels Luxembourg.

[8] Šabatka L., Wald F., Kabeláč J., Kolaja D. 2016. Vnitřní síly působící na styčník a globální analýza ocelových konstrukcí. Konstrukce. 3, 54-57.

[9] Wald F., *et al*. 2016. Benchmark cases for advanced design of structural steel connections. Praha: Česká technika - nakladatelství ČVUT. 2016, ISBN 978-80-01-05826-8.

[10] Da Silva Simoes L. 2008. Towards a consistent design approach for steel joints under generalized loading. Journal of Constructional Steel Research. 64, 1059-1075.

[11] Gödrich L., Wald F., Šabatka L., Kurejková M. and Kabelác J. 2016. To prediction of the connection deformation capacity by component based finite element method. Proceedings of the International Colloquium on Stability and Ductility of Steel Structures, SDSS, pp. 197-204, ISBN: 978-929147133-1.

[12] Sabatka L., Wald F., Kabeláč J., Gödrich L., Navrátil J. 2014. Component based finite element model of structural connections. Proceedings of the 12th International Conference on Steel, Space and Composite Structures, pp. 337-344, ISBN: 978-981090077-9.

[13] Jaspart J.P. 2002. Design of structural joints in building frames, 2002, Prog. Struct. Engng Mater. 4, pp. 18-34.

[14] Heinisuo M., Laine V., Lehtimäki E. 2009. Enlargement of the component method into 3D, Proceeding of Eleventh Nordic Steel Construction Conference, NSCC.

[15] Šabatka L., Wald F., Kabelác J., Kolaja D., Pospíšil M. 2015. Structural analysis and design using component-based finite element model. J. Civ. Eng. and Arch. 9, pp. 895-901.

[16] Wald F., Kwasniewski L., Gödrich L., Kurejková M. 2014. Validation and verification procedures for connection design in steel structures. 2014, Proceedings of the 12th International Conference on Steel, Space and Composite Structures. pp. 111-120, ISBN: 978-981090077-9.

[17] Abdalla K.M., Chen W.-F. 1995. Expanded database of semi-rigid steel connections. Computers and Structures. 56(4): 553-564, doi: 10.1016/0045-7949(94)00558-K.

[18] Jones S.W., Kirby P.A., Nethercot D.A. 1980. Effect of semi-rigid connections on steel column strength. Topics in Catalysis, 1(1): 38-46, doi: 10.1016/0143-974X(80)90007-3.

[19] HLAVÁČEK, Vladislav. 1976. Navrhování tenkostěnných ocelových konstrukcí: komentář k ČSN 73 1402, Praha: Vydavatelství Úřadu pro normalizaci a měření.

[20] Drosopoulos G.A., Stavroulakis G.E., Abdalla K.M. 2012. 3D Finite element analysis of end - Plate steel joints. Steel and Composite Structures. 12(2): 93-115, doi: 10.12989/scs.2012.12.2.093.