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Research paper

Experimental tests and FEM modelling of thermal effects of the welding process on steel anchor plates embedded in concrete

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Abstract: The paper discusses experimental tests and numerical analyses of welded steel anchor plates embedded in concrete. The purpose of the research was to assess thermal effects of the welding process on the plates and the concrete in which they were embedded. This assessment concerned both the effect of temperature on the material and the effects caused by the phenomenon of thermal expansion of steel. Parameters of the tested elements were selected so that they would reflect typical solutions applied in practice. The basic research consisted in welding gusset plates to anchor plates embedded in concrete and measuring temperatures at characteristic points of the tested elements. The FEM analysis model was built in a way ensuring that it would reflect the thermal effects accompanying the welding processes and the changing material parameters of the tested elements, i.e. steel and concrete, as well as various contact phenomena occurring between these materials. The need to take all these aspects into account considerably reduced the number of available numerical tools, limiting them to universal FEM programs, such as Abaqus and Ansys, where in the end Ansys was used in the analyses. The paper describes the testing procedures, FEM modelling, and presents the results of the research and analyses in question. The conducted research has shown that although the thermal effects of the welding process on the parameters of concrete are not significant, the thermal expansion of steel results in deformations of the plate sufficient to lead to damage to concrete layers surrounding the plate.

Keywords: FEM analysis, heat source model, steel anchor plate, thermal analysis, welding deformations

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1. Introduction

Anchor plates used to connect steel structure members with reinforced concrete structures are commonly used in practice. Taking into account the sequence of building construction and process-related aspects of carrying out respective works, the most common solution is to set the anchor plate in the formwork, embedding it in concrete together with the reinforced concrete structure, and weld the gusset plates to the anchor plate after the formwork has been dismantled. The research presented in this paper relates to the last of the above-mentioned stages, i.e. welding of the gusset plate to the anchor plate set in the reinforced concrete structure. Bearing in mind costs of the formwork and the need to keep it in good technical condition, the anchor plate is typically set inside the formwork, which means that ultimately the reinforced concrete member and the anchor plate concreted into it form one plane from the outside perspective [1–4]. Owing to this solution, there is no need to cut holes in the formwork and the subsequent finishing work around the anchor plate becomes easier. Thus, a favourable structural member is made, in which however, the surfaces of the plate that are in contact with the concrete cannot be assessed in terms of their condition after the welding process and at the same time, taking into account the thermal deformations related to welding, the tightness of the space between the steel and the concrete is disturbed. Solutions in which the plate is mounted outside the reinforced concrete member [5–11] and only anchor rods are concreted are less common. This solution is more difficult to implement and results in significantly worse tightness, which is of particular importance especially in the case of structures exposed to weather conditions.

In the literature, there are a number of research works devoted to various types of steel anchor plates embedded in concrete [1–11], however these works concern the load-bearing capacity issues in general and disregard the stage of welding the gusset plates to the anchor plates. In some of the research, the gusset plates were welded to the anchor plates before embedding them in concrete [1, 3, 5], which allowed eliminating key problems caused by the welding process. Nevertheless, it should be clearly noted that the assembly of complete anchor plates with previously welded gusset plates considerably differs from typical design solutions. Apart from construction-related aspects, one of the main reasons for welding the gusset plates after prior concreting of the anchor plates is the possibility to eliminate deviations in setting the plates in the reinforced concrete structure at the stage of welding connecting elements - assembly deviations of reinforced concrete structures are usually too large in the context of assembling steel elements. Welding of the gusset plates before embedding the anchor plates in concrete would significantly hinder the correct assembly of the steel structure later on.

This paper therefore addresses the issue of welding connecting elements to steel anchor plates embedded in concrete, assuming the most typical solution used in practice, i.e. a plate with an external surface levelled to the concrete surface. The purpose of the research was to assess the thermal effect of the welding process on both the plate and the material in which it is set, i.e. concrete. To better understand the phenomena considered here, it was assumed that in addition to laboratory tests, it is necessary to kg/m^3 conduct appropriate numerical analyses. While the experimental tests themselves did not pose any major problems, the correct numerical representation of the processes occurring in the tested elements was a greater challenge. It was necessary to build models that would enable taking into account thermal phenomena related to the welding process and the issues of complex interaction between two different materials (steel and concrete), as well as the mutual impact of welding on this interaction, and vice versa. These aspects made it necessary to use the versatile FEM software, which allows taking into account a number of different phenomena, but contrary to programs dedicated to selected expert issues, such as the welding process, do not provide the possibility of taking such processes directly into account. Issues related to the modelling of welding processes are described in the literature, see [12, 13], and some aspects of temperature distribution in welded materials [14]. Proper consideration of the thermal effects of the welding process on welded elements, as well as on the medium in which they are located, also required giving attention to issues related to heat flow in various media and between these media, the effect of temperature changes on this flow and, in general, phenomena related to the mutual interaction of various heated and cooling materials [15–17]. Some additional issues of numerical modelling of the phenomena presented in the paper were discussed in more detail in the diploma thesis [18].

2. Experimental tests

2.1. Specimens

The experimental tests covered a total of 21 elements with the first 5 being pilot tests used to identify the problem and adapt measurement methods accordingly, and the actual tests consisted of two series of 8 elements each - Table 1. The pilot series involved testing elements in which the anchor was made of a 16 mm thick plate measuring 200 × 240 mm, anchored in a 400 × 400 × 300 mm concrete block with the use of four ∅16 rods, and an 8 mm thick plate welded to the anchor plate with both-sided 6 mm thick fillet welds was the connecting element. The plate was anchored in such a way so as to enable dismantling of the anchor plate and assessing the space between the steel element and the concrete after completion of the tests. The anchor plate was therefore mounted on anchor rods with threaded ends and stabilized with nuts. Once the welding process was completed, the nuts were unscrewed, which allowed the dismantling of the plate and accessing the contact surface between the plate and the concrete.

Table 1. Summary of the tested specimens

Anchor plate surface	Pilot series	Main series	Supplementary series
Type 1 – without anti-corrosion protection	16 mm anchor plate – 3 pcs.	12 mm anchor plate – 2 pcs.	10 mm anchor plate – 1 pc.
		16 mm anchor plate – 1 pc.	12 mm anchor plate – 2 pcs.
		20 mm anchor plate – 1 pc.	16 mm anchor plate – 1 pc.
Type 2 – with anti-corrosion protection	16 mm anchor plate – 2 pcs.	12 mm anchor plate – 2 pcs.	10 mm anchor plate – 1 pc.
		16 mm anchor plate – 1 pc.	12 mm anchor plate – 2 pcs.
		20 mm anchor plate – 1 pc.	16 mm anchor plate – 1 pc.

In order to reflect real conditions of making such connections, some of the anchor plates were made without a layer of anti-corrosion protection and some with such protection. Intending to reflect real conditions of making such connections, the gusset plate was welded only after the anchor plate had been embedded in concrete and the concrete had achieved its target strength. C25/30 class unreinforced concrete and S235JR steel were used. In this case, the concrete was treated only as a medium in which the anchor plate was set and did not perform any other structural function, therefore reinforcement was not necessary. Furthermore, reinforcement would disturb the phenomena that were supposed to be observed during the tests, taking over part of the stresses caused by the thermal expansion of the plate heated by welding. Unreinforced concrete reacted more quickly and clearly to changes in the volume of the embedded steel element.

Thermocouples were used as the main measuring sensors, located under the anchor plate in areas where a considerable increase in temperature generated during welding was expected. Strain gauges were also used on the anchor rods to measure the strains of the rods resulting from the deformation of the anchor plate, however the results of these measurements are not discussed in this paper.

The pilot tests consisted of welding connecting plates to the anchor plate and recording the measurement results shown by the installed sensors. Having analysed the obtained results, an adjustment to the temperature measurement points was made, taking into account a rapid decrease in the recorded values along with an increasing distance from the heat source, i.e. the weld. Therefore, the distances of the temperature measurement sensors from the plate were reduced and the number of thermocouples was limited to 7 pcs. – Fig. 1.

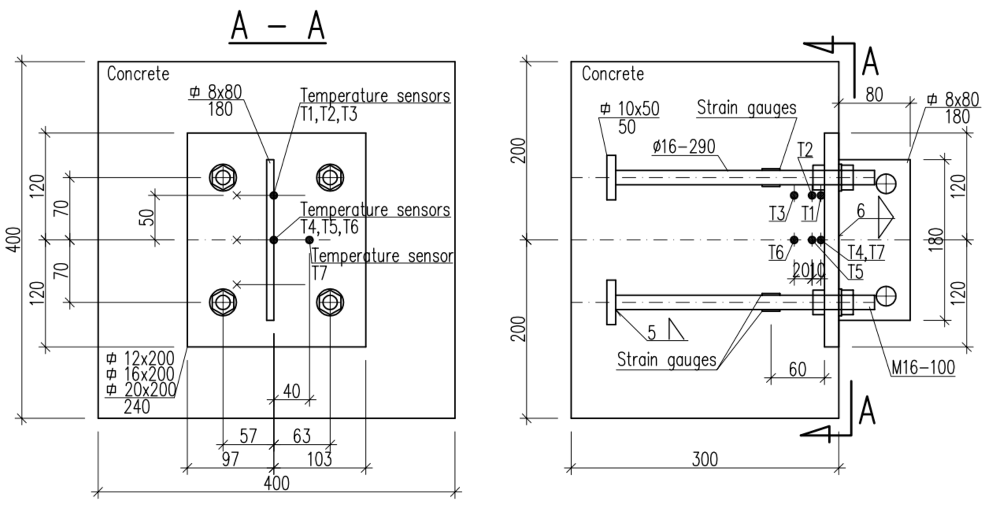
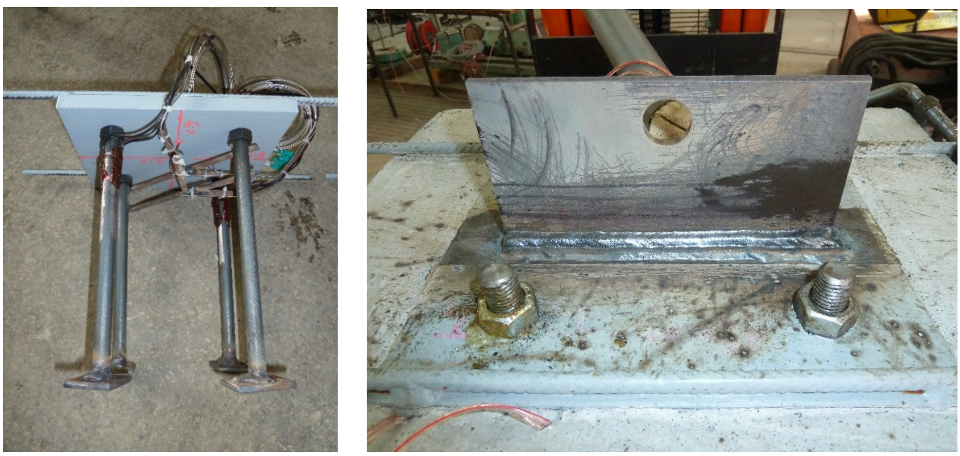


Fig. 1. Tested elements of main series

In the main series, 8 elements were tested – four with 12 mm thick plate and two elements with 16 mm and 20 mm thick plates, see Table 1. Similarly as in the pilot series, half of the elements were made with a paint coating on the side in contact with concrete, whereas the remaining elements without such a coating. Two layers of epoxy paint with a total thickness of 140 μm were applied. The tests, just as in the pilot series, consisted of welding gusset plates to anchor plates embedded in concrete and recording temperatures at the places where 7 thermocouples were installed, as well as deformations of the anchor rods. Double-sided fillet welds were made with three beads on each side, using the manual metal arc welding method (MMA) with ER146 (E462R11) electrodes. Typical parameters were adopted, i.e. welding current 100 A, arc voltage 24 V and electrode travel speed about 2.5–3 mm/s. Hence, the linear energy was in the range of 800–900 J/mm. Welding was carried out in line with the traditional method used in conditions of a construction site, i.e. layers of beads of small thickness were laid directly one after another. Making several weld beads without breaks to lower the temperature results in the introduction of a larger amount of heat in a short time interval and is therefore unfavourable from the point of view of thermal effects. At the same time, maintaining the appropriate material temperature at the welding spot reduces the risk of cold cracks. Welding with breaks allowing the material to cool down to a temperature of several dozen degrees would be disadvantageous from this perspective and would also significantly extend the time required to make the welds – in construction practice, welds of this type are usually made without longer breaks.

The results of temperature and deformation measurements were also recorded after the welding was completed, until the specimen reached the temperature of 50–60°C. In the supplementary series, tests were performed on 8 elements, including 2 pieces with a 10 mm thick plate, 4 pieces with a 12 mm plate, and 2 pieces with a 16 mm plate. The measurements and the test procedure were the same as in the main series. Representative specimens before and after the tests are shown in Fig. 2.



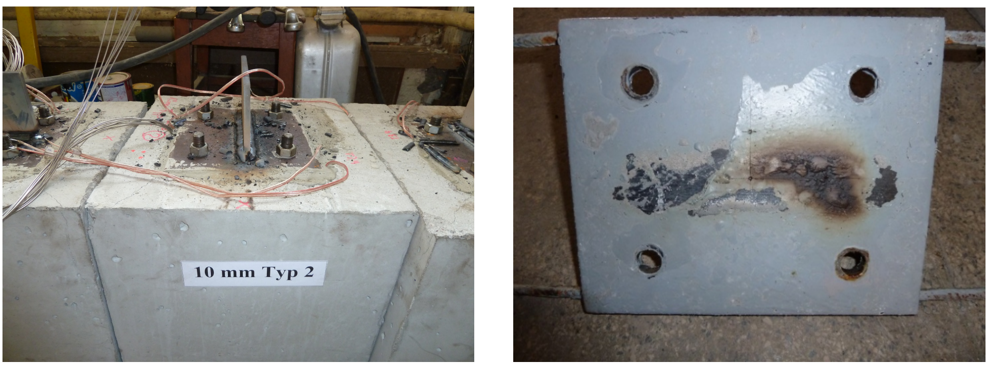
(a) (b)

Fig. 2. Tested elements before and after the laboratory tests; (a) Tested element with sensors, (b) Anchor plate with a welded connecting element

2.2. Experimental results

The aim of the conducted research was to assess the thermal effects of the welding process on the steel anchor plates embedded in the concrete and on the concrete itself. During the research, therefore, “continuous” measurements of temperatures recorded by installed sensors (thermocouples) were carried out, ending their recording after they reached a temperature of about 50–60°C, which, taking into account large dimensions of the tested elements accelerating heat dissipation, occurred after about 30 minutes from the end of welding. Deformations of the anchor rods were also recorded, however these measurements were not the main objective of the research. The phenomenon that was observed even without any sensors was concrete cracking in the corners (usually in two corners at one end of the anchor plate and in some cases in one corner) – Fig. 3a. It ought to be noted here that unreinforced concrete is sensitive to cracks in this type of corners, but before welding, apart from a single case, they were not observed. This means that they were mainly caused by the expansion of the steel anchor plate heated during welding. This is also confirmed by the observed relationship between the presence of ribbed mounting rods in the tested element, visible for example in Fig. 2, and the occurrence of the cracks. In the case of the tested elements with such rods (e.g. elements of the pilot series), such cracks were observed less frequently than in the elements without these rods. The rods increased the heat exchange surface of the steel element, making it to cool down faster and thus limiting the expansion of the anchor plate, which in consequence had a smaller effect on the surrounding concrete edges.

One of the aspects assessed in the research in question was the effect of welding on the anti-corrosion coating of the invisible surface of the anchor plate, i.e. the contact surface of the plate with concrete. As expected, the paint coating directly under the weld was damaged – Fig. 3b. This is obviously an unfavourable situation, in particular in the case of outdoor structures exposed to various environmental impacts. Bearing in mind the impossibility of ensuring adequate tightness of the space between the anchor plate and the concrete, especially taking into account the deformations of the anchor plate during welding, appropriate anti-corrosion protection should be used by applying paints resistant to elevated temperatures in the place of thermal effects caused by welding. The tests carried out show that the maximum temperatures on the surface of the anchor plate opposite the surface with welds did not exceed 400°C, so this should be the minimum thermal resistance of the paint applied to this surface.



(a) (b)

Fig. 3. Damage to concrete and anti-corrosion coating of the anchor plate; (a) Cracks in concrete at the corners, (b) Damage to paint coating at the anchor plate

From the point of view of the purpose of the research, the key results were the temperature measurements taken using thermocouples embedded in concrete. In the main tests, 7 sensors were used, placed under the anchor plate, at points allowing determination of the temperature distribution of the concrete during welding of the connecting element. Fig. 4 and 5 show the graphs of temperature measurements under the anchor plate with a thickness of 12 and 20 mm, respectively, and Fig. 6 shows a summary of temperatures for all 8 samples of the main part of the tests. In this case, graphs from the sensors recording the highest values were adopted, i.e. from thermocouples T1 or T4, located directly at the place of contact with the anchor plate.

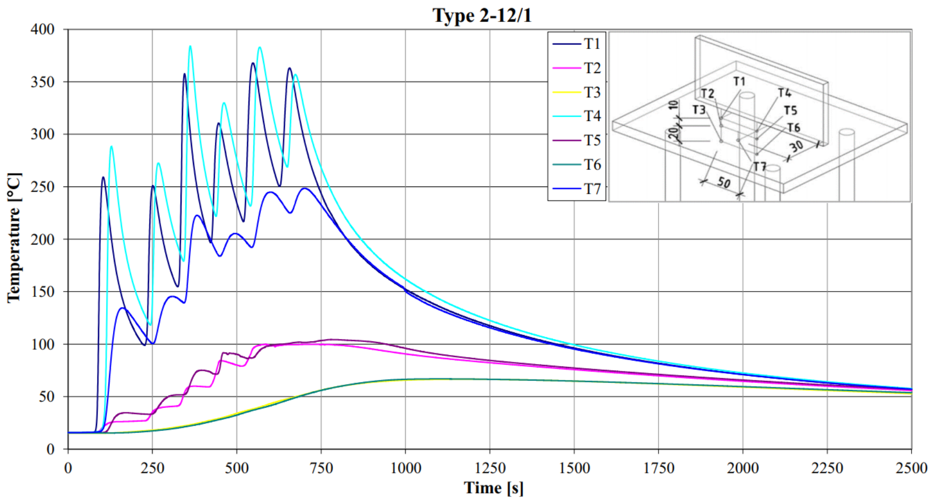


Fig. 4. Time – temperature relationships for the selected anchor with a 12 mm thick plate

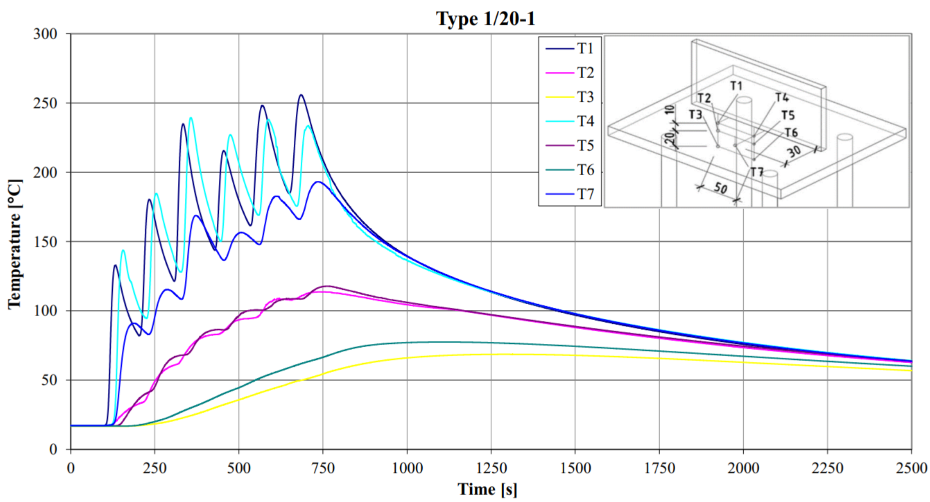


Fig. 5. Time – temperature relationships for the selected anchor with a 20 mm thick plate

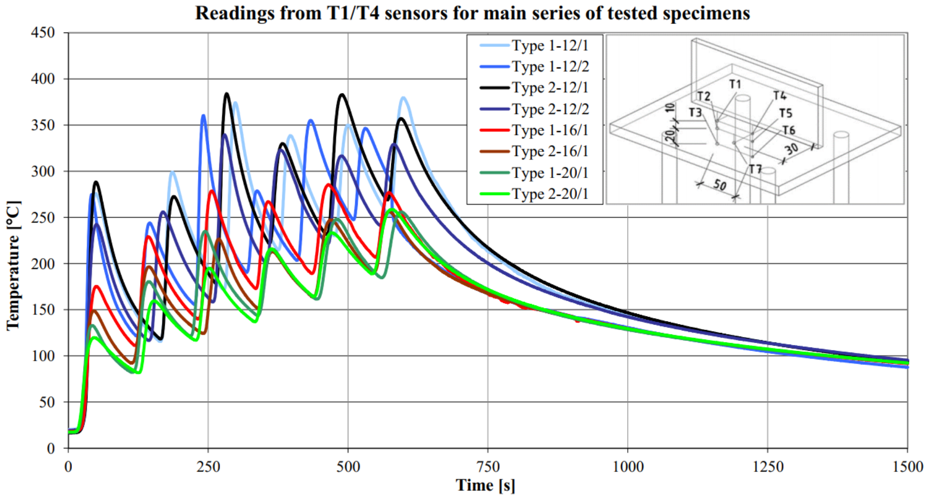


Fig. 6. Time – temperature relationships for main series of anchors

A general conclusion drawn from the research is that even with the assumed significant thicknesses of the welds, the highest measured temperatures in concrete do not exceed 400°C for 12 mm thick plates and 260°C for 20 mm thick plates. In the case of intermediate thicknesses, i.e. 16 mm, the highest recorded temperature values were slightly higher than for 20 mm plates and reached about 280°C. The impact of the anti-corrosion coating on the heating of concrete was generally negligible, i.e. no consistent relationship between the presence of the coating and the values of the measured temperatures was identified.

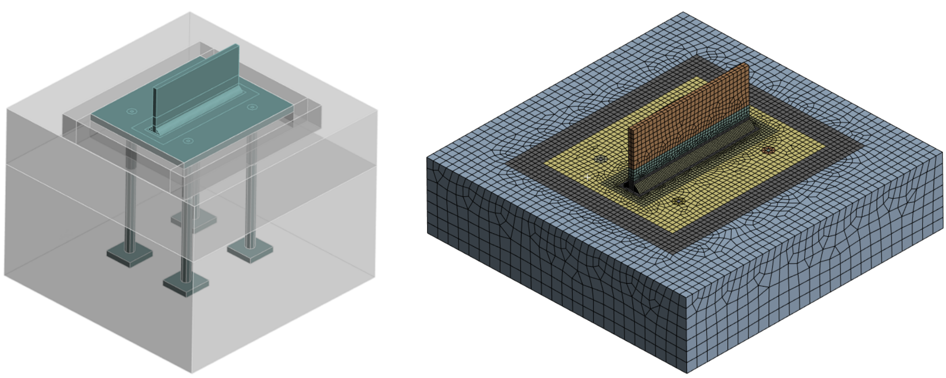
The highest values of the measured temperatures were several times lower than the temperature of the weld, which results from the very good thermal conductivity of steel, the cooling down of the plate from the outside by contact with the surrounding air, and heat removal from the inside by the concrete. The maximum temperatures to which the concrete heated up locally, not exceeding 400°C, do not pose a threat to this material. If we additionally take into account the fact that the sensors installed in the concrete to a depth of 10 mm did not register temperatures higher than 120°C, then even if it were reinforced concrete, such local and temporary temperature increases would not require any preventive or protective measures. Therefore, a conclusion can be drawn that the increase in the temperature of the anchor plate material and concrete caused by welding is not high enough to pose a threat to structural safety. Nevertheless, durability of the structure should be taken into account and anti-corrosion protection should be applied on the invisible surface of the plate, exposed to thermal effects, which would not be damaged by temperatures of around 400°C.

3. FEM analysis

3.1. FEM model

The universal ANSYS software was applied to perform the numerical analysis with the use of the finite element method, although in the context of modelling welding processes it would be more advisable to use more specialized software with integrated procedures dedicated to welding simulations. In this case, however, apart from the welding process, the issues of contact between different materials, i.e. steel and concrete, also required modelling [19]. In FEM modelling, the right selection of finite elements and division of the model into these elements play a key role. The analysed concrete-steel structure joint, with two interacting materials of considerably different physical-mechanical properties and dimensions, requires differentiating the finite element mesh in respective zones of the model. Finite elements with linear shape functions were adopted, mostly cuboidal 8-node elements, and in places that could not be modelled with cuboidal elements, elements of other shapes and with 4 to 6 nodes were used. A general principle was also adopted involving mesh refinement in zones where the greatest accumulation of the analysed phenomena and the greatest variability of such values as temperature and strains/stresses were predicted (i.e. in zones of large gradients of these values). The optimal mesh size of the FEM mesh was determined using the iterative method, with the aim of obtaining good convergence between the numerical and experimental temperature results. Finally, the mesh size of 2 to 16 mm was applied – with the smallest at the place of the weld. The size and complexity of the model, however, meant a very large number of finite elements and the resulting computational difficulties, which ultimately made it necessary to reduce the size of the FEM model and cut off a fragment of the solid that did not significantly affect the course of the observed phenomena – Fig. 7. The upper 100 mm part was left of a 300 mm thick block and convection was applied to the cut-off surface with a heat transfer coefficient of 100 $W/(m^2 \cdot K)$ for concrete and 2000 $W/(m^2 \cdot K)$ for steel, obtained using the iterative method.

From the point of view of the main purpose of the analyses, i.e. the assessment of the thermal effects of the welding process on the steel anchor plate and the concrete in which it is embedded, it is crucial to adopt proper parameters for the analysis. As a standard, in the first approach initial values are assumed, determined in conditions typical for the operation of most structures. In the case of an analysis concerning phenomena related to high temperatures, such an approach could, however, lead to incorrect results. It was therefore assumed that the parameters whose values depend on the material temperature will be validated based on a comparison of the results obtained from the thermal analysis with the available laboratory test results.



(a) (b)

Fig. 7. FEM model; a) Geometry of the complete model, b) FEM mesh

The first parameter whose value could affect the results of thermal analysis was the density of steel at different temperatures. Based on the work [15], it was assumed as decreasing nonlinearly from 7850 kg/m^3 at room temperature to 6900 kg/m^3 at temperatures exceeding 1600°C. According to the EN 1993-1-2 standard [20], the thermal conductivity coefficient was assumed as decreasing linearly from 54 W/(m·K) at 20°C to 27.3 W/(m·K) at 800°C and higher. The standard [20] was also the basis for assuming the specific heat capacity, the value of which changes considerably at the phase transformation temperatures, i.e. 700-800°C. The specific heat capacity at 20°C is 425 J/(kg·K), at 735°C it increases to as much as 5000 J/(kg·K), and then decreases to 650 J/(kg·K) at temperatures above 900°C. As regards concrete, its density was assumed based on material tests conducted at the stage of experimental tests described in section 2 - the value of 2330 kg/m^3. The thermal conductivity coefficient was assumed in accordance with the EN ISO 10456 standard [21] as for reinforced concrete (due to steel anchor rods and a steel frame supporting thermocouples), in line with the EN 1992-1-2 standard [22], taking into account the dependence of this coefficient on temperature. The value of the coefficient changes linearly from 2.5 W/(m·K) at 20°C to 1.6 W/(m·K) at 300°C (the concrete temperature did not exceed 300°C). Also the specific heat capacity was assumed according to the standard [22], with the value varying from 900 J/(kg·K) at 20°C to 1050 J/(kg·K) at 300°C.

Convection and radiation phenomena have a significant impact on the temperature distribution in elements made of materials such as steel or concrete, causing heat losses through external surfaces of such elements. As far as the case of concrete is concerned, the temperature of external surfaces generally oscillated around 25°C and was higher only in the direct vicinity of the steel anchor plate, whereas in the case of steel, the situation was complicated by significant temperature differences during the welding and cooling process. In the case of concrete, the convection coefficient of 13 W/(m^2·K) was assumed, based on the information contained in [16] and this value did not apply to the cut-off surface of the concrete block, which was described above. As regards the steel plates, the convection and radiation coefficient was adopted based on the paper [14] with a value changing along with temperature from 10 W/(m^2·K) at a temperature not exceeding 200°C to 157 W/(m^2·K) at a temperature of 1600°C. In the context of the effect of welding on the temperature of the concrete in which the anchor plate was embedded, the thermal conductivity between steel and concrete is of key importance and it depends on factors such as surface roughness, air spaces between the two materials, stresses, and surface temperature. In this case, the information contained in the work [17] was used and the values of this coefficient were adopted that were changing along with an increase in temperature and that depended on the plate thickness, varying from 250 W/(m^2·K) at the beginning of the welding process to 50–65 W/(m^2·K), respectively, for plates 12–20 mm thick in the final phase of welding. During the cooling phase, this coefficient increased to the maximum value of 200 W/(m^2·K). On the side contact surfaces of the steel and concrete, the value of 100 W/(m^2·K) was assumed.

The most difficult issue was to properly take into account the welding process itself. With various available models of the heat source in the welding process, having first conducted a series of analyses, a model of a moving heat source was adopted in the form of a surface limited by an oval with a larger diameter of 10 mm and appropriately selected power density. Based on the guidelines included in the book [23], different power intensities (35–39.5 MW/m^2) were applied, depending on the thickness of the anchor plate (from 12 to 20 mm), which, taking into account the used load application areas (on average about 57 mm^2) and the heating time (60–75 s) adapted to individual weld beads, with 3 beads on each side of the plate, allowed supplying such an amount of energy (900 kJ) that gave thermal effects consistent with the results of laboratory tests.

Apart from the thermal analysis, the developed model was also used for strength analysis, therefore it was also important to correctly reflect the strength parameters of steel and concrete. The anchor plate was made of S235 grade steel, so the yield point was assumed accordingly and due to the significant temperatures, in accordance with the EN 1993-1-2 standard [20], a corresponding reduction thereof was applied. An elastic- plastic model of the material with a bilinear stress-strain relationship was adopted. The modulus of elasticity was also assumed to be reduced in accordance with the standard [20]. However, both in the case of the yield point and the longitudinal elasticity modulus, it was necessary to introduce a certain deviation from the standard assumptions and the reduction was carried out only up to a temperature of 800°C, which was aimed at preventing zero values of these parameters, at which numerical analysis would not be possible. Considering the fact that this deviation concerned only the welds and small fragments of the base metal directly next to the welds, this procedure did not lead to a noticeable disturbance of the results. The thermal expansion coefficient and Poisson's ratio were assumed as required for this type of steel, with a temperature-dependent reduction according to the information stemming from the literature [24].

C25/30 concrete was used for the tested element and the compressive strength (25 MPa) as well as Young's modulus (31 GPa) were adopted for this material in accordance with the EN 1992-1-1 standard [25]. The thermal expansion coefficient and the elastic-plastic material model characterized by a bilinear stress-strain relationship were assumed according to the same standard.

At the steel and concrete contact surfaces, frictional contact was assumed according to the so-called Penalty formulation that allows for a small amount of material penetration (in reality, there is no such penetration), eliminating at the same time the problem of significant local reactions occurring in the model of contact without penetration. The friction coefficient was assumed at 0.2. In the strength analysis, an additional external load on the connecting plate was assumed to simulate the action of a beam attached to the anchor plate.

3.2. FEM analysis results

The first and the most important stage of numerical analyses related to temperature distributions in the tested element, caused by the thermal effect of the welding process. As described in subsection 3.1, the welding process was numerically modelled in such a way so as to reproduce the welding of connecting elements to the anchor plate with double-sided fillet welds 6 mm thick on each side and assuming that each weld consists of three beads applied without additional process breaks. Fig. 8 presents a comparison of the results obtained from numerical analyses with the results of experimental tests of a specimen with a plate 12 mm thick. The convergence of results can be assessed as very good, but with a slight deterioration as the thickness of the plate increases. In the case of temperatures measured in the contact surface of the plate with the concrete, the differences are at a level not exceeding 2–3%. In the case of places farther away into the concrete from the plate, the percentage differences are greater both during the heating and cooling time. These discrepancies may partly result from the deviations of the distance of the sensors from the weld, which in the tested specimens could slightly differ from the designed distances, while the temperatures in the model were recorded at the designed locations. The second reason is the difficulty in ideally reflecting the phenomenon of thermal conductivity of concrete in the numerical model, which in the case of sensors "isolated" by concrete from the heated anchor plate, is of great importance. Concrete consists of, among other things, aggregate with diversified packing and size of grains, which in the case of a larger body of concrete can be averaged, but locally causes differences in conductivity. As long as this was reflected in the laboratory test results, it could not be taken into account in the model and in consequence led to the observed discrepancies.

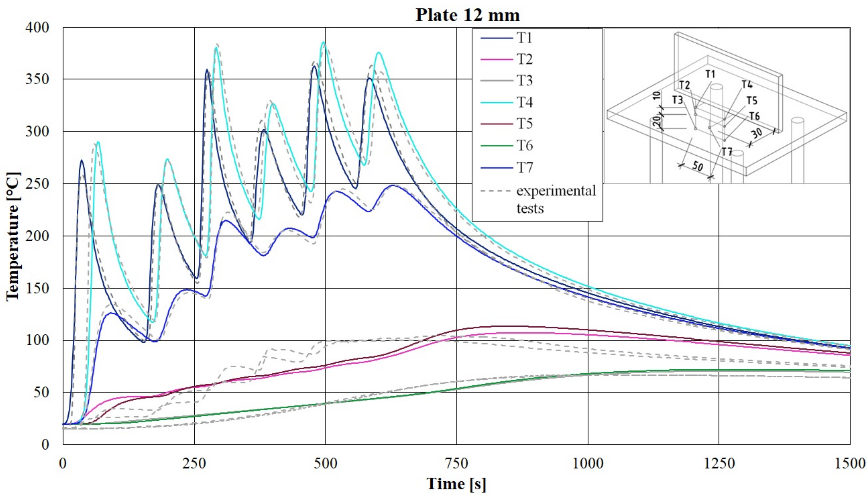


Fig. 8. Comparison of the results of numerical analyses and experimental tests – plate 12 mm thick

In order to assess the correctness of the adopted method of modelling the thermal effects of the welding process on steel anchor plates and the concrete in which they are embedded, the temperatures obtained for the three analysed plate thicknesses, i.e. 12, 16 and 20 mm, were compared, but with the same thermal load parameters characterized by the power intensity, the place of application of the thermal load, the duration of respective cycles, and the breaks between them. Fig. 9 presents a comparison of temperatures obtained at one of the points at the contact of the plate with the concrete, directly under the weld. The conclusions are consistent with those obtained during the experimental tests, i.e. along with an increase in the plate thickness, the temperature decreases.

The strength analysis was carried out in multiple steps, with changing thermal load, divided into three phases: welding, cooling, and static loading. Finite elements used in thermal analysis (SOLID278) were replaced with SOLID185 elements with three degrees of freedom, using linear shape functions and enabling nonlinear analysis with large deformations. The concrete material model does not allow cracks to develop. The results of the static analysis are not discussed in this paper, but they confirm the additivity of residual stresses from welding and stresses caused by the external load.

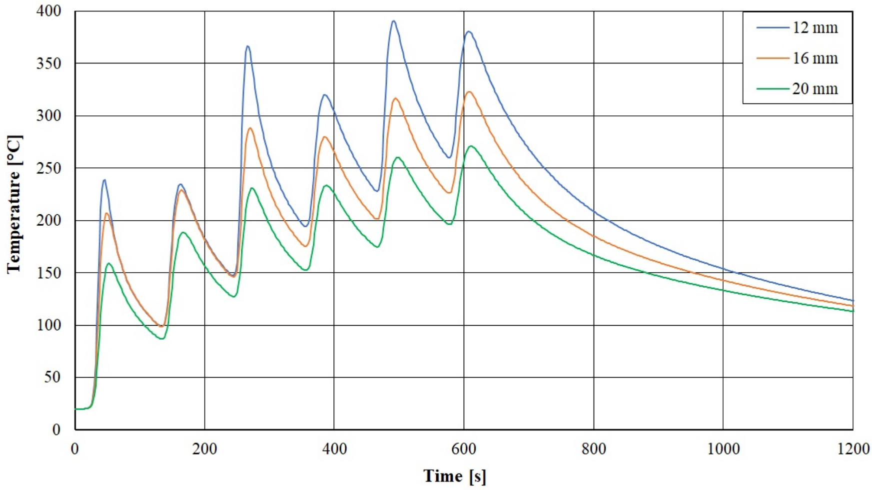
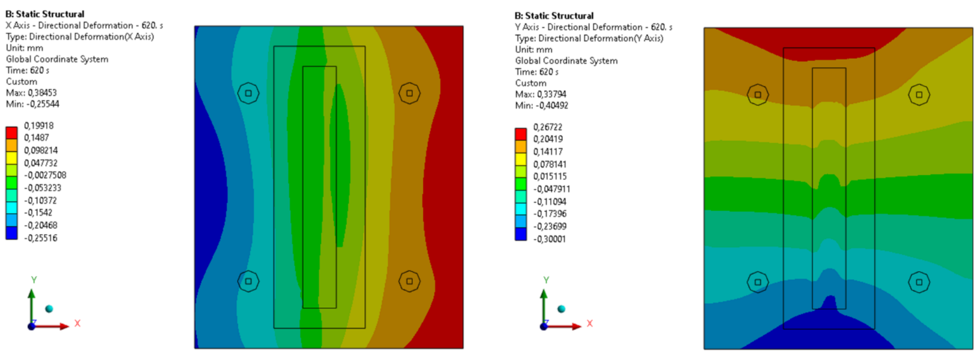


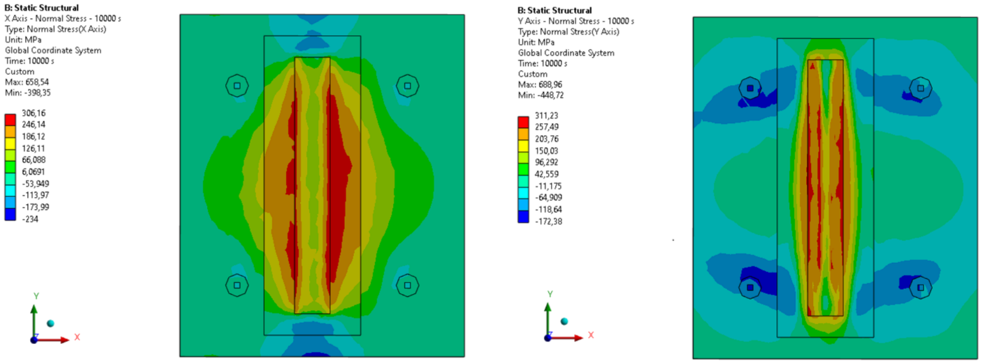
Fig. 9. Comparison of numerical analyses results for different plate thicknesses

From the perspective of the research objective, of the greatest importance are the changes in the dimensions of the anchor plate resulting from welding, which brings large amounts of heat into the material. Fig. 10 shows a map of deformation of the steel plate after welding. As can be seen, they are equal to approximately 0.2 mm in each direction, which means that when the plate is tightly fitted to the concrete, the side edges of the plate will press on the concrete, which may result in the cracks discussed in section 2 (Fig. 3a). After the element cools down, the anchor plate deformation disappears to a large extent, but the cracks in the concrete will remain. Also stresses were observed during the analysis and the most important observations concern the values of these stresses after cooling - Fig. 11. As one can see from the stress maps, their values near the weld significantly exceed the yield strength.



(a) (b)

Fig. 10. Anchor plate deformation maps; (a) Deformations in the XX direction, (b) Deformations in the YY direction



(a) (b)

Fig. 11. Welding stress maps after the anchor plate has cooled down; (a) Stresses σx, (b) Stresses σy

4. Concluding remarks

Laboratory tests and numerical analyses carried out show that in the case of typical solutions of steel anchor plates embedded in concrete, the maximum temperatures to which the concrete locally heated up on the surface of contact with steel did not pose a threat to this material. Moreover, placing the sensors in the concrete at a depth of as much as 10 mm resulted in approximately a 3-fold reduction in the recorded temperatures, which means that even if the concrete were reinforced, such local and temporary temperature increases would still not pose a threat. However, the aspect of the structure durability should be taken into account and the invisible surface of the anchor plate exposed to thermal effects should be covered with such anti-corrosion layer that will not be damaged by the temperatures generated during welding. In similar anchor plates solutions, these temperatures can reach up to 400°C. Attention should also be paid to the issue of thermal expansion of welded steel elements and stresses that will occur in the concrete under the pressure caused by the expanding, heated plate. This is a particularly important issue in the case of anchor plates embedded in concrete, where the pressure on the concrete around the perimeter of the plate can lead to concrete cracks in the corners. This circumstance should therefore be taken into account when designing such anchor plates and such solutions should be applied that minimize the expansion of the heated plates and, consequently, concrete cracking. It is therefore necessary to limit the amount of heat introduced during welding by using proportionally small weld thicknesses, compared to the dimensions of the anchor plate, or by implementing breaks in welding process. The concrete should be reinforced at the corners of the anchor plate, and it is also worth considering rounding the corners of the anchor plate.

The analyses carried out confirm the usefulness of FEM for modelling the phenomenon of thermal effects of the welding process on steel anchor plates embedded in concrete. The results obtained on the basis of the numerical model are characterized by good convergence with the results of the experimental tests. The analyses confirm that it is possible to use universal FEM tools for this type of studies, e.g. ANSYS, which allow comprehensive modelling of elements made of different materials and taking into account different phenomena, which would not be possible in the case of specialised programs designed for modelling welding processes.

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Badania doświadczalne i modelowanie MES wpływów termicznych procesu spawalniczego na marki stalowe osadzone w betonie

Słowa kluczowe: analiza MES, analiza termiczna, marka stalowa, model źródła ciepła, odkształcenia spawalnicze

Streszczenie:

W artykule omówiono badania doświadczalne i analizy numeryczne spawanych marek stalowych osadzonych w betonie. Tego typu elementy konstrukcyjne stanowią klasyczne rozwiązanie stosowane w połączeniach konstrukcji stalowych z żelbetowymi, umożliwiające prawidłowe scalenie elementów wykonanych w różnych reżimach tolerancji. W literaturze opisano szereg badań takich elementów, przy czym generalnie dotyczą one zagadnień nośności, a pomijane były technologiczne aspekty montażowe związane z procesem spawania elementów węzłowych do blach osadzonych w betonie. Właśnie spawanie jest tym procesem, który zapewnia uzyskanie odpowiedniej dokładności montażu konstrukcji stalowej, łączonej ze znacznie mniej dokładnie wykonaną konstrukcją żelbetową. Przeprowadzono więc badania, których celem była ocena wpływów termicznych procesu spawalniczego na marki oraz na beton, w którym były one osadzone. Ocena ta dotyczyła zarówno wpływu temperatury na materiał, jak też efektów wywołanych zjawiskiem rozszerzalności termicznej stali. Parametry elementów badawczych dobrano w taki sposób, aby odzwierciedlały typowe rozwiązania występujące w zastosowaniach praktycznych. Badaniami doświadczalnymi objęto 21 elementów, przyjmując typowe, z praktycznego punktu widzenia, marki z blach o grubościach od 10 do 20 mm, z elementami węzłowymi 8 mm spawanymi spoinami pachwinowymi. Polegały one na spawaniu blach węzłowych do osadzonych w betonie marek i pomiarach temperatury w charakterystycznych punktach elementów badawczych. Prowadzono także pomiar odkształceń prętów kotwiących oraz wizualną ocenę zjawisk zachodzących w trakcie spawania i stygnięcia elementów, przy czym w niniejszym artykule nie omawiano wyników pomiarów tensometrycznych. Model do analizy MES zbudowano w taki sposób aby możliwe było odzwierciedlenie zarówno efektów cieplnych towarzyszących procesom spawalniczym, jak również zmieniających się parametrów materiałowych elementów badawczych, tj. stali i betonu oraz zjawisk kontaktu pomiędzy tymi materiałami. Konieczność uwzględnienia ww. aspektów znacznie ograniczała możliwe do wykorzystania narzędzia numeryczne, zawężając je do uniwersalnych programów MES, takich jak Abaqus i Ansys – w opisywanych analizach wykorzystano Ansys. W artykule opisano procedury badawcze, modelowanie MES oraz przedstawiono wybrane wyniki badań doświadczalnych i analiz numerycznych. Przeprowadzone badania wykazały, że o ile wpływy termiczne procesu spawalniczego na parametry betonu nie są znaczące, to rozszerzalność termiczna stali skutkuje na tyle dużymi odkształceniami marki, że może prowadzić do uszkodzenia otaczających markę warstw betonu i zjawisko to powinno być uwzględnione w procesie projektowania tego typu rozwiązań konstrukcyjnych.

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