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BRACE ARRANGEMENT, NODE STIFFENING AND CONNECTION OF SPRINGS OF STEEL TUBULAR ARCH BRIDGES

RASPORED UKRUĆENJA, UKRUĆENJE ČVOROVA I VEZA S KRAJNJIM OSLONCIMA ČELIČNIH CEVASTIH LUČNIH MOSTOVA

Philippe VAN BOGAERT

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1 INTRODUCTION

Steel tubular arch bridges are highly appreciated for their aesthetical value and elegance. This is due to many factors, such as the curvature of members, tubes being considered more ‘simple’ than other sections, low surface area compared to equivalent open sections, smooth transition of light shading, avoidance of sharp edges between shaded and lighted surfaces and also more technical features. This has also contributed to the success of tubular structures in buildings, roofs, spatial structures, telescopes, transmission lines and masts and offshore structures [1]. In addition, tubular structures show high torsional stiffness, lower wind pressure and low weight.

However, the nodes are its counterparts. The junction of 3 or more intersecting tubes is complicated, both to design and to fabricate. In some bridges, the nodes are made from cast steel and then welded to connect the meeting tubes. Other showcases have welded nodes, requiring precise cutting of individual members as well as large use of welding material. The nodes are determining the costly character of tubular bridges and their degree of complexity. However, they also clearly show the transfer of forces between members. Hence, the elegance of this type of structures is highly determined by the flawless character of the connections of members.

Particular types of node are the arch springs. They are of paramount importance since they transfer the various reactions from the bridge superstructure to the abutments. The most important reaction is the arch thrust force together with the vertical reactions. Whether bending moments are transferred to the infrastructure at the arch springs depends on the use of hinges or clamping. The fabrication of hinges certainly is more complicated than complete clamping of arch springs. In most cases, the concept of the arch spring connections corresponds to clamping and bending moment transfer.

Since the torsional capacity and stiffness of circular sections are rather high, large torsion clamping may be expected. Experience has demonstrated that torsional clamping may not be entirely efficient and that small angular rotations may occur, in case of clamping by pre-stressing bars. As an alternative, clamping by steel-concrete connectors may be considered. This has recently been applied to 2 bridges, the general idea being to attribute a certain type of connector to resist one or several types of reactions. To underpin such connections, three preliminary experiments, including strip and stud connectors have been carried out to demonstrate the failure mechanisms. From the first loading a composite action of steel and concrete exists. Failure of strip connectors is due to exceeding compression of the surrounding concrete, followed by high pressure in the contact area between both materials and final failure due to yielding the steel strips as reinforcement of the composite cross-section. Failure of stud connectors occurred prematurely due to concrete splitting at the contact area of the stud shaft. This is avoided by normal reinforcement, provided it interacts with the studs. Application of the test results by scaling to the case of a recently built steel arch shows that its connection with internal headed studs does not contribute to torsion stiffness. However, scaling of the test results demonstrates that the arch spring almost behaves as in free torsion condition.

Professor Em. Dr. Ing. Philippe Van Bogaert – Ghent University (Belgium)

2 EXAMPLES OF RAILWAY TUBULAR BRIDGES

2.1 Merxem Street truss tubular arch

The Merxem Street Bridge was built to carry the high-speed railway line from Brussels to Amsterdam and is located immediately to the North of the city of Antwerp [2]. It is part of the North-South link below the city of Antwerp. The crossing with Merxem Street is located in-between two more important steel bridges across the Albert Canal and the Ijzerlaan. They have spans of 117 and 56 m respectively.

At this particular site, the high-speed line is located parallel to domestic tracks. The latter cross the Merxem Street by a double vaulted brickwork arch, dating from 1910 and spanning the 18 m wide local road. The brickwork vaults are well preserved and show historic value. Since the building permit covered the whole North-South link, including large transformations of many listed heritage structures, special care had to be taken to preserve the function and the view on older structures. This was in particular the case for the Merxem bridges. Hence, both a functional and aesthetic bridge of a particular character, complying well with historic building had to be designed.

In view of these conditions, several alternatives have been considered for this bridge. Obviously, vaulted arches include heavy abutments and especially the view on the arch crown should be kept. Therefore, the span of the new bridge had to be sufficiently large in order not to hide these important parts. In addition, arches can be highly compatible with the basic function of the vaults. This almost excludes traditional beam bridges having a span of approximately 27 m. A first type of alternative is to build a steel tied arch with slender lower chord. The disadvantage of this type resides in the higher level of the superstructure. The existing bridge being located below the tracks, a new structure of larger altitude disturbs the site in general. Hence, a structure below the tracks is to be preferred.

Figure 1. Merxem street tubular arch bridge

Since there are few fine examples of tubular arches, this type of structure could certainly be considered. The advantages are clear, provided the arch itself would consist of a single tube as can be seen in figure 1. The horizontal distance of the single tube to the front of the brickwork reaches 8 m, the interesting parts of the vaults being seen from the street level. In addition, the concrete deck is supported by inclined tubes, constituting members of a tubular truss. This arrangement also contributes to contrast with the massive nature of brickwork vaults. At the same time the modern steel truss, fabricated with modern welding processes renders a flawless shape, contrasting highly with the rough texture of old brickwork. This contrast of textures and shapes may be the best way of dealing with the challenges of combining old and new bridge structures.

2.2 Woluwe Lane framed tubular arch bridge

The Woluwe Lane Bridge allows 3 tracks to cross an important access road to the North of Brussels and for a new railway line, connecting Antwerp to Brussels airport [3], to enter the central reservation of the motorway. A particular characteristic of the bridge is that both lateral tracks are located at a distinctly higher level than the central track. This is because the latter has a different direction, since it is connected to a bend and is joining the Eastern circular railway around Brussels in the direction of the European Parliament. The location of this bridge made authorities to expect a particular design. Because of the difference of levels of the tracks and the subjacent roads, a tubular arch bridge was designed. This multi-level bridge consists of 2 almost parallel tubular arches, including tubular strut members, supporting the upper level tracks. The bridge deck of the central track is mostly suspended to both lateral arches. Due to the unbalanced action of the arches, the latter are slightly eccentric with respect to the upper level track axis. The arch span equals 78 m, its rise being limited to 9 m. Each concrete deck slab is supported by 2 tubular members, connected to an arch. Hence, these members show bending moments at the connections, the bending stresses being considerably larger than the stress due to axial compression. The Woluwe Lane Bridge is seen in Fig. 2.

As there is no direct connection of the arch top with the concrete upper decks, a short **double strutting** system has been provided. This connection is rather heavy and provides larger bending stiffness to the structure. Particular elements of this type of bridge are the arch springs, which may be hinged or clamped. In general, clamping of arches is easier to build, by using a base plate, connected to the concrete infrastructure by pre-stressing bars. Difficulties may arise in finding a workable manner to close the arches while controlling the effective arch thrust force. In the case of Woluwe Lane bridge, as the arches were assembled on site from 4 parts, the thrust force has been measured during jacking of adjacent parts. The steel tubes were welded on the building site, with appropriate procedures, to compensate welding shrinkage and allow fitting of the main tube parts.

Figure 2. Woluwe Lane Bridge

2.3 Truss versus frame arrangement

Both bridges, commented before, have a different system to connect the upper floors with the main arch tube by bracing members. Typically, Merxem street bridge has a triangular **in-**plane arrangement of the braces whereas the brace members of Woluwe Lane Bridge constitute a framework with the floor deck and the main tube. Since both bridges have different span, a comparison of both structures is irrelevant. Hence, as a first step, the structure of Merxem Street was transformed into an equivalent bridge, having framed bracing tubes.

The definition of an equivalent bridge mainly applies to obtaining identical stress states in the various members of the structure, both the main arch tube and the bracing members. This can only be achieved by successive trial values of the characteristics of the tubes. In this, mainly the vertical loads due to dead weight and traffic loads have been considered. Thermal effects, as well as horizontal loads are not included in this comparison. The first type of loading has little consequence in this type of bridges, whereas the second type of loads in mainly diverted directly to the abutments. The bridges being considered are integral bridges, all concrete decks being fixed to the abutments.

Fig 3 shows the 2 types of structures, the triangular arrangement in the cross section being identical, since transverse connection of the deck slab to the single main tube has to be ensured. In addition, it should be mentioned that the values shown further are factored and concern ultimate limit state. Fig. 4 shows the comparison of normal forces and in-plane bending moments for both cases. Clearly, the envelope values of the normal arch compression force are quite different. Whereas for triangulated arrangement, the normal compression force is decreasing from the arch springs towards the top, in the framed arrangement, it remains rather constant. The former is due to the diversion of normal forces by the inclined braces. The arch compression force increases by some 27%.

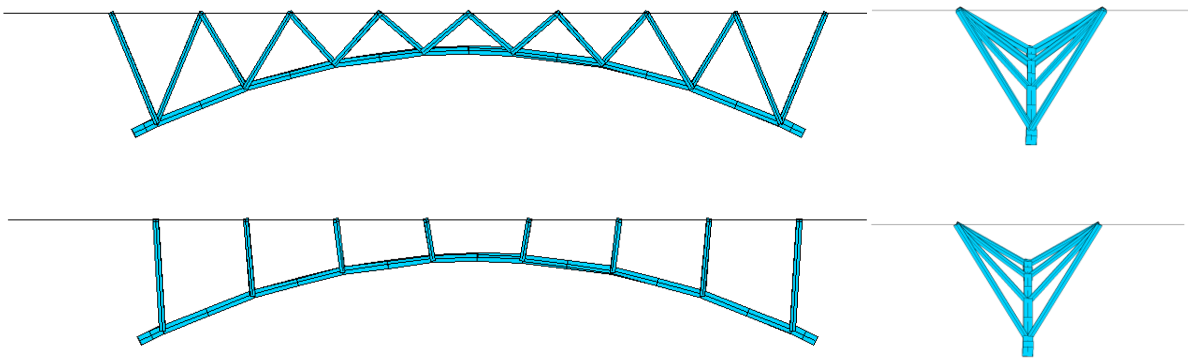
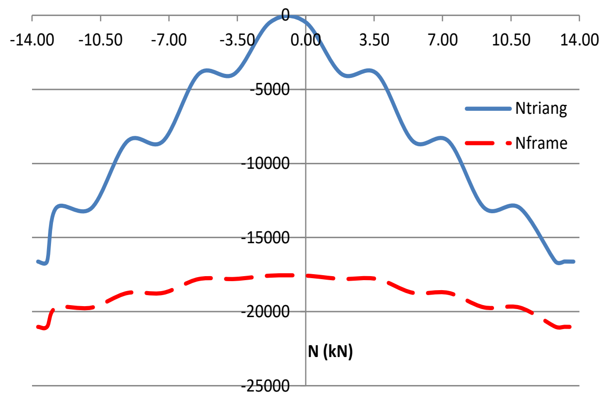


Figure 3. Triangulated (top) and frame arrangement of bracings (below)



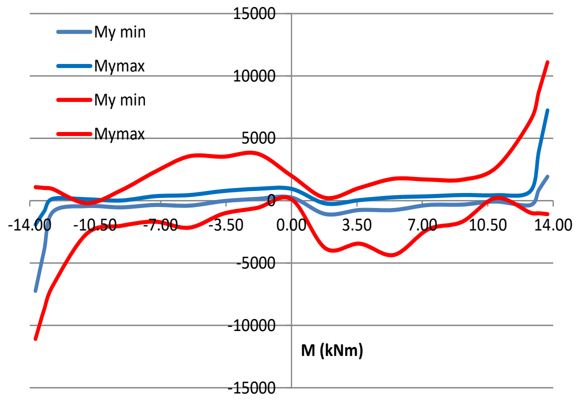


Figure 4.: Normal arch force (left) and bending moments (right)

Fig. 4 (right) also displays the in-plane bending moments in the arch tube. Maximum and minimum values have been derived. The diagrams are not entirely symmetrical, since asymmetrical loading is more relevant and there is no point in repeating calculations of both sides. In both cases the largest bending is found at the arch springs, where bending is increasing rapidly. Evidently the outside braces transfer larger bending to the main tube as the clamping moment is increased by some 50%. Nevertheless, also in other parts of the main arch tube bending moments are larger for the frame arrangement than for the triangulated alternative. To have some idea about the magnitude, the most relevant bending moments are up till 6-times larger. In addition, other maximum values are appearing near quarter-span of the bridge, which is easily explained by the influence line of clamped arches. The general conclusions concerning the difference between both alternatives is that triangular brace arrangement performs distinctively higher, since the bending is reduced and the arch compression is lower near the arch top.

Similar diagrams are obtained for the brace tubes. However, the number of framed braces is half of the number in the triangulated alternative. On one hand, there is the observation that for triangular arrangement the number of brace members is double of that for the frame. On the other hand, vertical forces are deviated to inclined members, which are obviously larger than the original vertical force. Hence, in spite of the fact that there are twice as much braces for the triangular arrangement, the normal force is of the same order of magnitude.

In addition, the bending moments are seriously larger in the frame alternative. The multiplication factor may rise up to 6 for the most relevant values. However, the increase of stress resultants does not adequately quantify the most important characteristic, steel mass. The aforementioned comparison results in a difference of steel mass varying from 25 to 30% in favour of the triangulated alternative, which is in accordance with similar findings for tied arch bridges [4].

3 NODES

3.1 K- and T-nodes

Truss arrangement of braces, connecting a tubular arch to an upper bridge deck necessarily requires the nodes to be of the K-type. The latter is generally more complex than the simple T-type node, which is used in the frame alternative. This may constitute sufficient ground for the frame option. Both types of finished nodes are shown in Fig. 5.

The nodes constitute important discontinuities in the stress flow of the main arch tube. This causes stress concentrations near the weld toes of the nodes, mainly in the arch tube. These stress concentrations may seriously decrease the load carrying capacity of the structure, although they are not systematically considered at ultimate limit state of strength. In addition, the stress concentrations certainly reduce the fatigue strength, which mainly applies to road and railway bridges. Fatigue resistance, rather than ultimate limit state has determined the design of both the Merxem Street as well as the Woluwe Lane bridges.

The stress concentrations are due to heavy welding, the geometric discontinuity, the introduction of various stress resultants from the brace tubes and also the local deformation of the main arch tube member. The latter is the effect, shown in fig. 7 and is certainly due to bending moments at the nodes. Fig. 7 is the result of detailed FEM-modelling of several nodes as in [5]. Both normal force and bending moment from the smaller brace tube tend to disturb the circular cross-section of the tube, introducing a complex three-dimensional stress-state, since it introduces a circumferential bending of the tube wall.

A method has been developed to assess accurately the stress concentrations in tubular connections [5], including those introduced by shear and torsion. This method allows independent assessment of stress concentrations (and so-called hotspot stresses) due to the various member forces and moments, derived from simple wireframe calculation models. The advantage of this method resides in the fact that it may be applied to any type of K- or T-joints, whatever the complexity may be.

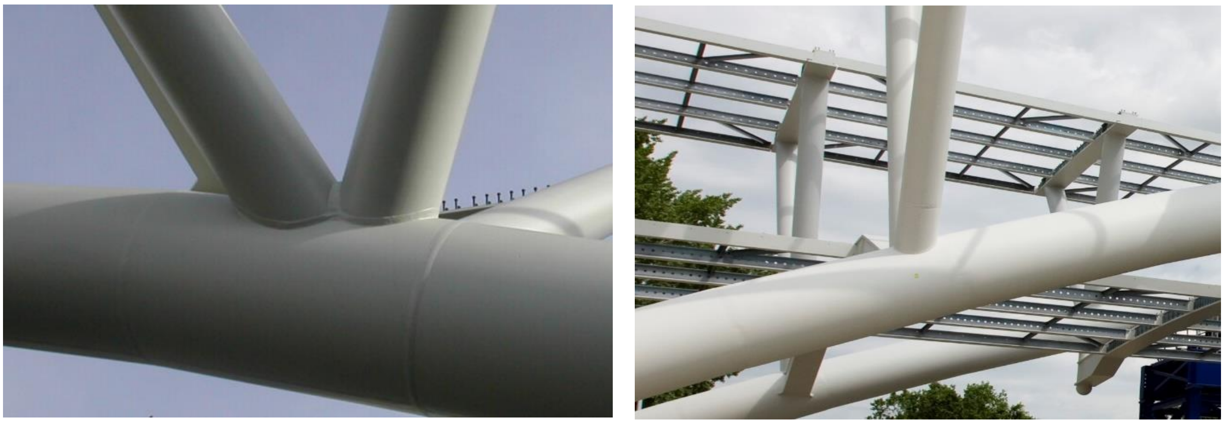


Figure 5. K-node (left) T-node (right)

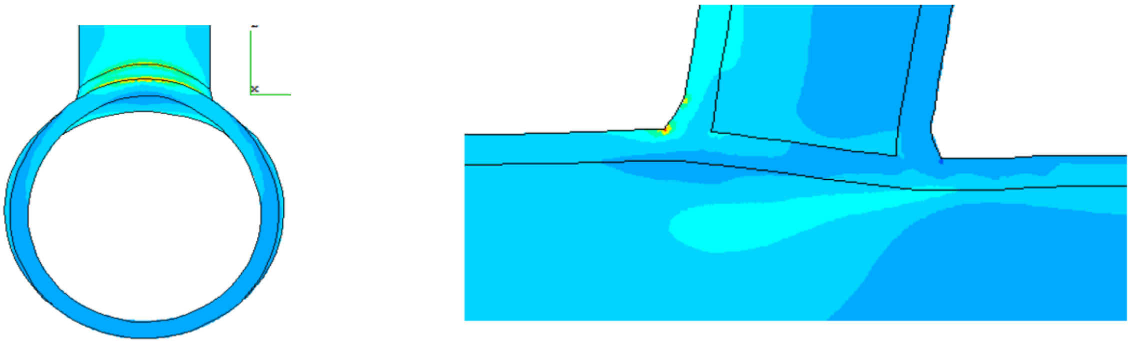


Figure 6. Local deformation

This work has confirmed that stress concentration factors may arise in K-joints up to 2.2, whereas for T-joints the concentration is mostly milder to 1.85. These figures are taken from a few examples worked out for both Merxem Street and Woluwe Lane bridges.

3.2 Node stiffening

The existence of stress concentrations may inspire designers either to generally increase the wall thickness of tubes, both in braces and main arches. However, this would result in uneconomic use of material and poor performance. Increasing the diameter of the main arch tube intensifies the effect of local deformation and certainly lowers the aesthetic character of the structure. The main arch chord is meant to be a strong member, but the braces need sufficient volume as well, since they are essential in the load transfer. In both bridges discussed, the ratio of outward diameters equals 2, which resulting from [6] probably is an optimum value. Regarding the node stiffening, local reinforcement can be preferred.

Since the effect of welding cannot be eliminated, the single option consists of reducing the local deformation and distortion part of the stresses, as shown in Fig. 6. One way of obtaining local reinforcement is to supplement the arch tube outside wall with curved reinforcing plates at the exact location of the node. The increase in thickness provides larger cross-sections and is preventing deformations as in Fig. 6. However, this type of reinforcement also decreases the aesthetical value of the tubular arch, since the smooth surface of members is interrupted, much of the pleasant appearance of tubular arches being due to perfect transition of the members.

Hence, internal stiffening of the main arch tube is the optimum alternative. This type of stiffening requires sufficient internal space of the main tube and to weld diaphragms inside the tube. These diaphragms increase the stiffness of the member and are most efficient to prevent the local deformation. The diaphragms may be open or full. The first option is superior for steel conservation.

Fig. 7 shows the internal stiffening for both bridges of Merxem Street and Woluwe Lane. The fundamental difference between them is clearly seen since for the K-node (Fig. 7 left) the diaphragms are located at the intersection of the brace axis with the surface of the main tube. This is the location where the normal member force is transmitted. This arrangement was inspired by the fact that in the truss arrangement bending moments are relatively lower and both meeting brace members are intersecting at the connection. In T-nodes larger bending is expected for frame arrangement (Fig. 7 right). Hence a double diaphragm is used, located exactly below the outside ribs of the brace tube. This should introduce adequately the bending moments in the main arch tube.

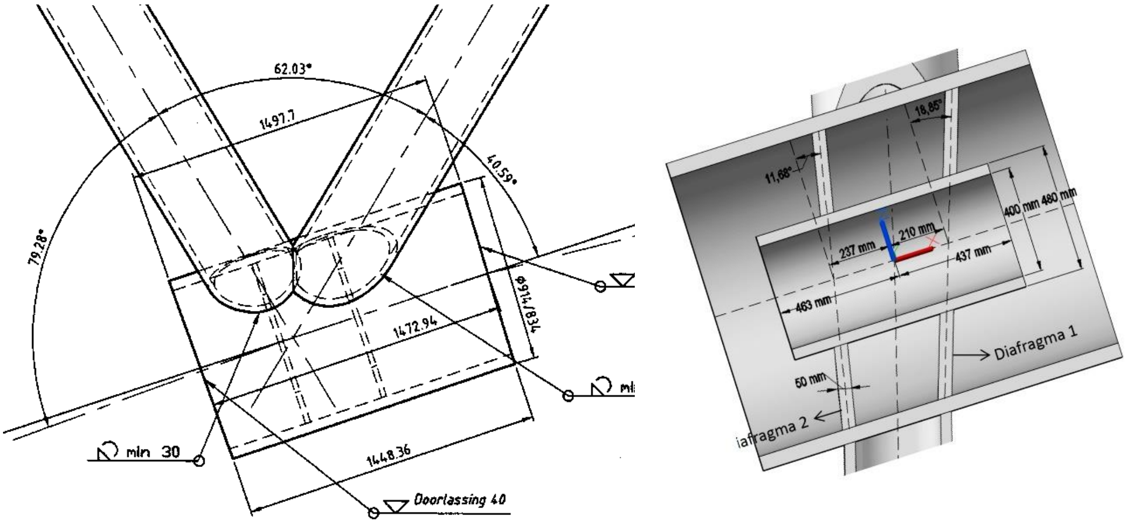


Figure 7. Node stiffening by diaphragms

Since in the case shown in Fig. 7 (right) a second system of brace tubes is provided, to connect the central lower bridge deck to the main arch, the diaphragms are slightly sloping towards this brace. In addition, calculations showed that excessive stress appears at the inner edges of the diaphragms themselves. The stress level can effectively be reduced by short tubes welded to the inner opening of the diaphragms. These small tubes constitute additional flanges to the ribs of the diaphragms. However they are not connected to the main tube and do not disturb the transfer of forces and moments. Fig. 8 shows the preparation of a double diaphragm node. Since nearby butt welds connect the node to the other parts of the member, Fig. 8 may be convincing that the solution is also practical.

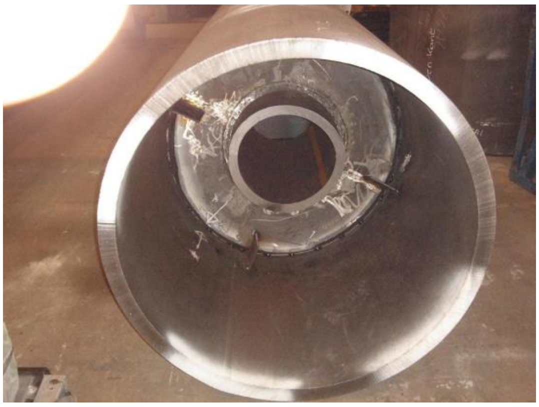


Figure 8. Fabrication of internal stiffening

3.3 Optimum location of diaphragm stiffening

A series of numerical simulations and tests [6] confirmed that diaphragm stiffening is really effective to lower stress concentrations, the latter being reduced by factors between 1.7 and 1.8. This corresponds to the evident location as in Fig. 7 the centre plane of the diaphragm corresponding exactly to the centre of the outward rib of the brace tube. However, this does not necessarily correspond to the optimum location for stress concentrations or fatigue resistance. To reduce the former, the location of Fig. 9 shows better performance, the centre plane of the diaphragm corresponding to the outside surface line of the brace tube. This may constitute the optimum location for static loading.

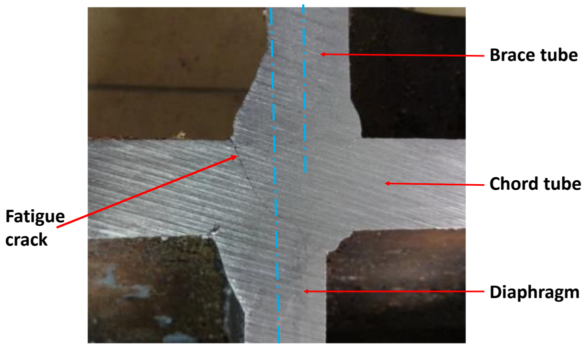


Figure 9. Fatigue crack

However, fatigue resistance may be the main issue for bridges. With the location of Fig. 9 the effects of the opposite welds clearly is too close, causing interference of both possible crack locations, as can be seen in Fig. 9. The actual fatigue resistance is lower than for the evident arrangement. The results show that the largest fatigue resistance is obtained if the centre plane of the diaphragm coincides with the inner surface plane of the brace tube. This avoids the cracking pattern where both welds of the brace to the main tube and the root of the diaphragm weld are interacting.

4 SCALED TESTS

The experience of Woluwe Lane bridge has demonstrated that torsional clamping may not entirely be efficient and that small angular rotations may occur, in case of clamping by pre-stressing bars [7]. As an alternative, clamping by steel-concrete connectors may be considered. This has recently been applied to 2 bridges, the general idea being to attribute a certain type of connector to resist one or several types of reactions. A picture of this principle is shown in Fig. 10. The strips, welded to the 1.6 m diameter tube are equipped with headed studs and resist tensile force due to bending, whereas the rings effectively resist normal thrust compression force. Shear is transferred from the steel tube to the concrete abutment by the interior concrete and limitation of contact stresses. As for the torsion moment, it is resisted by both the welded strips and the headed studs. In the case of Fig. 10 the torsion moment was rather low and the torsion stiffness was not a real issue to verify the torsion capacity of a steel tube connected in concrete by 2 different types of connectors. In particular, the strip and headed stud types were to be tested. In addition, it was expected that the tests would give useful indication on how to improve the concept of reinforcement at the connection of the tube and concrete abutment.

Figure 10. Arch spring with connectors

4.1 Test aims and setup

The 3 tests that were conducted are reported here in a summarized version. More details may be found in [8]. The aim of the test is to apply a pure torque to a steel tube, encased at its lower end in a concrete block and to determine the failure value of the torque, the rotation angle and the failure pattern. Since these tests were intended as a first approximation and mainly to detect what the failure mechanisms are in this type of connection, the samples were scaled and limited to 3. Possibly the results may constitute an introduction to a wider program.

The setup, shown in Fig. 11, consists of a truncated parallelogram concrete slab of 1.2 by 1.2 m and 0.15 m thickness with a minimum of reinforcing rebars. To reduce the torque magnitude and to use limited auxiliary equipment, the tests are a scaled situation of the real situation by a factor of approximately 10. Hence, the concrete itself was to be scaled as well and is in fact mortar.

Vertical UPN 160 profiles of S 235 are encased in the concrete slab and are serving as anchorage points to materialize torsion. Finally, each sample included a vertical steel tube 50/8 S 235 welded to a horizontal UPN 80 S 235 profile. The latter allows its connection by load cells to the vertical UPN profiles and thus the application of 2 equal horizontal forces, which act as a torsion moment at the base of the vertical tube.

Figure 12. Test setup

4.2 Test 1 no connectors

The first test was intended as a trial and also aimed to detect whether any **natural bond** would exist between the encased part of the steel tube and the concrete slab. Natural bond would certainly influence the resistance of the connection. In addition, it is unsure whether natural bond would simply add to the connector's resistance or have a more complex influence.

Hence, the steel tube end was encased in the concrete at **50 mm depth**. This corresponds to the diameter of the steel tube. In real structures it is unlikely that a steel tube would be encased to 50 mm only, although a depth equalling the tube diameter is a common value. Consequently, any natural bound would have been a disturbing factor for further testing.

The code [9] does not recognize any natural contribution to bond. In the present case the concrete strength obtained just before testing was rather low, since temperature was low with frost overnight. This conducted to results for $f_{ccub150}$ from Table 1 and an average resistance of 11.2 MPa. This table also includes the values for tests 2 and 3.

Table 1. Concrete resistance $f_{ccube150}$ (MPa)

Sample	1	2	3
Test 1	**11.4**	**11.0**	**11.2**
Test 2	**17.4**	**16.6**	**17.4**
Test 3	**19.1**	**17.4**	**16.4**

During the test it became evident that natural bond was inexistent. The weight of the equipment of load cells, hooks, turnbuckles and chains made the connection fail immediately.

Figure 12. Broken cylinder

The torsion capacity due to bond was limited to 0.051 kNm. In addition, after removing the steel tube, a cylinder seemed to have broken from the internal concrete, as can be seen in Fig. 12.

The torsion capacity of this broken cylinder is very low and can be estimated between 0.004 and 0.008 kNm. Obviously, neither the failure of the cylinder nor bond can be of influence to the following tests.

4.3 Test 2 strip connectors

In the second test the steel tube end was equipped with 4 50 mm long strips with a cross section of 8*8 mm. Thus the steel tube reached a depth of 75 mm, since the last 25 mm was also encased in the concrete slab. This corresponds to the arrangement of Fig. 10, without stud connectors.

The connection should have failed for a design value of the torque equalling 0.613 kNm. The latter corresponds to yielding of the strips due to shear, taking into account the yield stress of 235 MPa. However, the yielding does not necessarily correspond to failure. It introduces large deformation and may initiate other failure mechanisms. The graph of Fig. 13 shows the relation of the angular rotation versus torque.

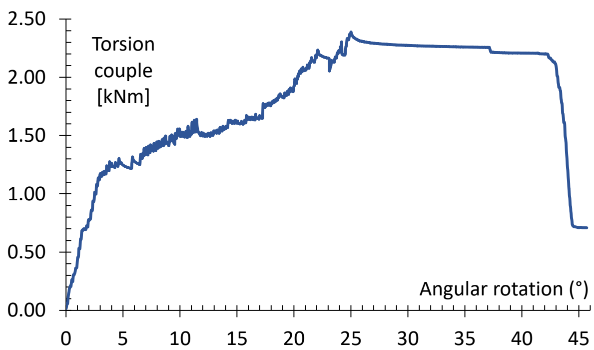


Figure 13. Load-time diagram test 2

In this diagram several failure steps can be found. They will be analyzed further. Already one may see that after the first linear phase, the curve deviates from the linear part and becomes flatter for a value of the torsion moment of about 1.3. This demonstrates that a new mechanism must be reached. From a torsion moment of 1.7 kNm the curve again becomes steeper and reaches a maximum for a torque of 2.39 kNm. After reaching the maximum the curve continues first with a horizontal part and rapidly growing angular rotation and finally descends steeply. The conclusion must be that 2.39 kNm is the failure moment.

Some observations were made after this test. As Fig. 14 shows, the strips were excessively bent, although no real sign of cracking or fracture was found at the strip base. This demonstrates that failure of the steel strips alone did not occur during the test. In addition, after removal of the steel tube, the strip ends seem to have been fixed in the concrete, as can be seen in Fig. 15. Apart from some crushing at the edges, there is but little damage to these prints. This might suggest that a bending mechanism of the strips has existed, consisting of a support at the concrete side and a certain degree of clamping at the end of the steel tube.

Figure 14. Excessive bending of strips

Figure 15. Prints made by strips

4.4 Test 3 stud connectors

In the 3rd test, an identical tube is equipped with 2 headed stud connectors. The bolts, simulating studs, were welded to the tube and were intended to be encased at a depth of 25 mm. A detailed picture of the bolts and tube can be seen in Fig. 16. The total length of the bolts equals 60 mm, including the head of 13 mm. The steel grade differs from classical stud connectors, albeit the value of f_u = 400 MPa is close to the usual 420 MPa.

The predicted value of this connection was based on either the crushing of the surrounding concrete or shear of the connector shaft. As for the torsion moment, the lever arm would be determined by the tube diameter. Thus the predicted value reached 0.98 kNm. As for the previous test, a diagram of the torque versus the angular rotation was determined. The graph is shown in Fig. 17.

Figure 16. Stud connection

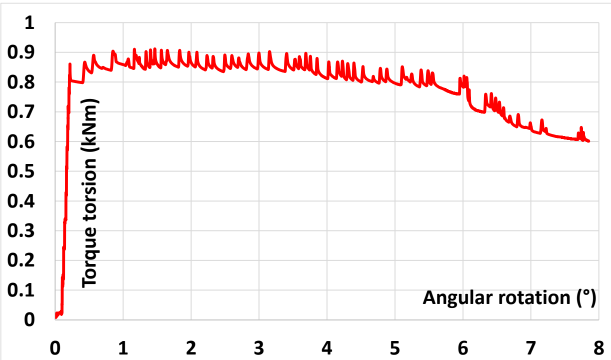


Figure 17. Torque versus angular rotation

In the first part of the diagram a linear relation of the torque with the angular rotation is ending at the maximum value of the torsion moment of 0.825 kNm. This is definitely lower than the predicted value. From there the curve starts descending while the rotation increases rapidly. The torsion moment decreases till it reaches a

Figure 18. Concrete failure of stud connection

value of 0.7 kNm. During this phase cracks start appearing at the concrete surface and the latter is spalling. After this phase the curve further descends more rapidly and apparently a new mechanism is starting. The rotation can further increase for a lower value of the torque, while the concrete spalling is further increasing. Finally, cracks become general and the torque drops from a value of 0.55 kN at the end of the curve. This is the final failure point. After removal of the spalled concrete the end situation becomes clear as in Fig. 18. The studs are not broken, but heavily bent. There is no apparent cracking of any of the steel parts and thus the concrete part must have failed.

5 ANALYSIS OF TEST RESULTS

5.1 Strip connectors

The aim of this section is to provide acceptable explanation of the experimental results and thus try to derive a failure condition and the robustness of both types of connections. Obviously, these tests do not pretend completeness and the main aim is to detect the failure mechanisms.

The first discontinuity in the diagram of Fig. 13 corresponds to T = 1.327 kNm. Considering composite bending as well as the fact that the ends of the strips have been fixed at the lower parts of the concrete, as seen in Fig. 15, the loading condition of the strips may be considered similar to a one-side clamped and one-side simply supported beam.

Assuming such a scheme, the largest bending moment appears at the clamped edge. The scheme may be considered as a RC-beam, the reinforcement being equal to the steel strip. As for the concrete cross-section, this may have a height of 8 mm, as the steel strip and a width of 3-times this value. These characteristics are a reasonable assumption, given the geometric parameters of the connection.

The resisting bending moment of the RC-beam may now be determined by assuming the concrete stress in the 8 * 24 mm area to reach 3 f_{cm}, according to the appropriate code [10] the strength value for local contact compression. This resisting bending moment corresponds to a value of the shear force of 15.8 kN for each strip and thus a torsion moment of T = 1.327 kNm. This almost exactly corresponds to the summit of the first linear part of the diagram of Fig. 13.

From these data one may conclude that the first discontinuity in the diagram is due to reaching concrete compression failure of the RC-section of a strip and the surrounding concrete. In these conditions, the actual contact stress equals may still increase before reaching 3 f_{cm} = 31.6 MPa. This also applies to the steel stress, since its value reaches 184.5 MPa, well below the yielding stress.

However, the slope of the diagram now reduces, while the torsion moment can still increase. The actual concrete contact stress can still rise, until the concrete compression equals 3 f_{cm}. At this point the torsion moment equals 1.727 kNm, which corresponds exactly to the end of the second part of the diagram of Fig. 13. Hence, from this point the concrete at the concave side of the excessively bended strips has failed laterally. The diagram then shows again a steeper third part of the curve.

As mentioned before, the steel stress of the RC-cross section can still be increased up to the failure value of 400 MPa. For the one sided clamped and one side simply supported beam this occurs for a total lateral force of 21.67 kN, corresponding to a torsion moment of 2.39 kNm, which corresponds well to the maximum value of the diagram of Fig. 13.

Although various assumptions have been made, it is believed that all 3 characteristic points of the diagram of Figure 14 are thus explained and that composite action of the strips, due to heavy lateral compression and friction, is the evident way to explain the results of test 2. At any rate, the failure torsion moment of 2.39 kNm is considerably larger than the torque of 0.615 kNm which would be the yield point of the strips. Hence the connection would surely have failed earlier, should there not be any interaction with the encasing concrete.

5.2 Headed stud connectors

As for the strip connectors, the 3 characteristic points of the torsion moment versus angular rotation diagram are being assessed. However, the diagram of Fig. 17 is completely different from the one of the strip connectors. The first linear part stops at the maximum of the curve and must be considered as the failure load. This maximum value of the curve corresponds to a torque of 0.83 kNm. According to the appropriate code [3] the shear resistance of the studs is the lower of both values P_{u1} and P_{u2} determined as:

$$P_{u1} = \frac{0.8\ f_u\ \pi\ d^2}{4} \tag{1}$$

$$P_{u2} = 0.29\ \alpha\ d^2\ \sqrt{f_{cm} E_{cm}} \tag{2}$$

The second formula, corresponding to concrete failure proves to be determining and renders a failure shear force of 19.575 kN or a torsion moment of 0.98 kNm, exceeding the maximum value of the curve. Obviously, the connection has less resistance.

Figure 19. Early concrete surface cracking

In section 5.4 the surface cracking at early stage during the test was mentioned. This is shown in Fig. 19. This type of cracking may be explained by local compression introduced through the stud shaft and subsequent splitting of the upper part of the concrete. This is in accordance with the code [9] since the equations (1) and (2) are no longer valid when studs are arranged in a way that splitting forces occur in the direction of the slab thickness, unless reinforcement is provided. Hence, since no reinforcement was interfering with the cracks, the studs should have been located at larger depth in the concrete slab.

The limitation of splitting reduces the maximum shear of stud connectors and is expressed by equation for T_{uspl} (3).

$$T_{uspl} = \sqrt{2}\ \eta\ f_{cm}\ r_{stud}\ \ell_{act}\ D \tag{3}$$

In this η is a geometric factor, estimated at 1.14 and l_{act} is the active length of the stud for splitting. The latter was estimated at 53 mm. In equation (3) the value of D must be taken as the tube diameter plus the connector shaft length. The ultimate torsion moment takes the value of 0.825 kNm and complies with the experimental value.

Once the concrete has been split entirely, roughly half of the encasement subsists and it may be assumed that the connectors are still active. Again, it must be concluded that at this stadium a composite action must exist. Should the studs act independently, the cross-section yields for a bending moment of

$$M_y = \frac{4\ R^3\ f_y}{3} \tag{4}$$

The maximum shear force corresponding to the bending moment (4) may be calculated. It must be multiplied by the tube diameter plus the length of a single stud to obtain the corresponding torsion moment of 0.489 kNm. This is considerably lower than the second characteristic point of the curve. The value of T = 0.71 kNm corresponds almost exactly to the ultimate bending moment for a RC section, the rebar being the stud cross-section and the concrete part being an area of 30 * 10 mm, in a similar manner as for strip connectors.

The end of the curve corresponds to a large angular rotation as can be seen in Figure 19. The stud heads are still encased in the concrete and are able to resist a tensile force corresponding to the yield stress of the studs. Taking f_y = 320 MPa this results in the value of T = 0.533 kNm.

5.3 Deformation capacity

In the diagram of Fig. 20 both curves have been combined and limited to the lowest values of the angular rotation. The difference between both curves is **striking**, since the rotations are totally different.

From the data the specific rotation angle can be determined. They have been summarized in Table 2. The table also indicates that the experimental torsion angle (in rad/m) is measured from the concrete surface to the location of the stud stiffness GC, assuming the specific torsion or the connection with the strip.

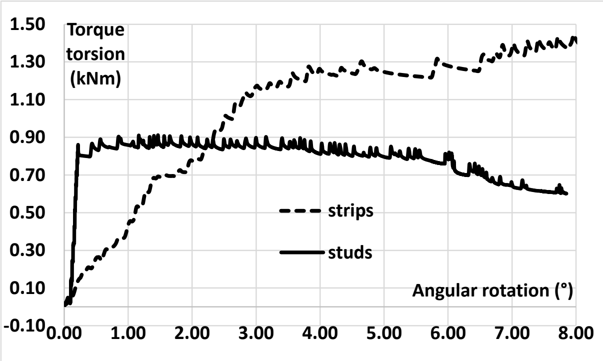


Figure 20. Diagrams for strips and studs combined

Table 2. Torsion angular rotations

Connector type	Strips	studs
Angle Rad/kNm	**0.0370**	**0.00872**
Torsion stiffness kNm²/rad	**0.5411**	**2.8685**

Table 2 confirms that strip connectors, reputed to be of the stiff type are more flexible than studs, which are generally considered as flexible. In addition, the values may be compared to GC = 37.6 kNm² of the steel tube, which is respectively 70-times and 13-times stiffer than the strip- and the stud connection.

5.4 Arch spring rotation

The Moerbrug bridge, near to Bruges, is a steel tied arch, with springs located below the chord members. The connection to the concrete abutments is similar as in Fig. 10, a combination of strip and stud connectors. Unfortunately, as shown in Fig. 21, the connectors are located inside the steel tube. Hence, their resistance is exclusively due to concrete strength.

During casting of the bridge deck, the temporary bracing, connecting both arches was removed too soon. This temporary bracing can be seen in Fig. 22. Due to this prematurely removal, the concrete casting resulted in lateral displacement of the arch tops by 40 mm. The latter cannot be explained by the missing bracing alone, since calculations show that the lateral displacement should be limited to 21.04 mm.

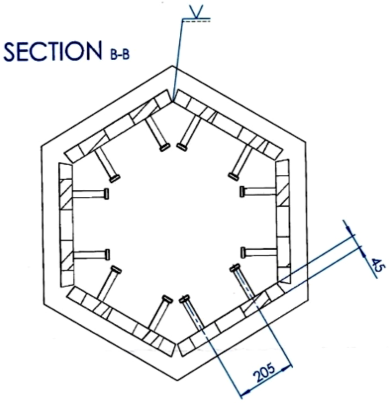


Figure 21. Arrangement of connectors

Several causes for the excessive deformations of the arch tops can be considered. One of them is the lack of torsion stiffness. Hence, it was suspected the lack of torsion stiffness, possibly combined with lateral movement of the connection, may be a source of excessive deformation.

Figure 22. Bridge with temporary bracing

For the loading case that caused the lateral displacements, the torsion moment equals 130.4 kNm. Should the axial rotation of the arch springs not be prevented, it takes a value of 0.0026 rad. If the angular rotation of stud connection is scaled to this case by the parameters of stud diameter, active length, concrete strength, tube diameter and number of studs, a value of the rotation of 0.000244 rad is found. However, the torsion may also result in the separation of an internal cylinder as in Fig. 3. If the torque for breaking this cylinder is also scaled to the bridge size, the failure value corresponds to 15.46 kNm.

These data lead to the conclusion that the internal studs have little contribution to the arch rotational clamping. Hence, the strips encased in the concrete abutment have the largest contribution to the limitation of arch spring rotations.

According to similitude law, the rotation stiffness value for strips from table 2 can be scaled to the actual bridge, including the parameters of length t, diameter D, concrete strength f_{cm} and number N of strips, according to (5) (p is for real structure, m for model)

$$\theta_p = \theta_m \, T_p \, \frac{t_{strip\ m} \, D_m \, f_{cm\ m} \, N_m}{t_{strip\ p} \, D_p \, f_{cm\ p} \, N_p} \tag{5}$$

the angular rotation becomes 0.008637 rad, which is larger than if there would be no torsion clamping at all. This indicates that the connection with internal studs may be ineffective for torsion.

6 CONCLUSIONS

It was implied that the load carrying capacity of tubular arch bridges with truss arrangement of the brace tubes connecting the upper bridge deck to the arch is larger than the equivalent frame arrangement. This is due to lower normal compression force at the arch crown and lower clamping moment at the springs for the truss alternative. Both of these characteristics also apply to the brace tubes. Equivalence of the load carrying capacity is reached if 25 to 30% more steel is used for the frame alternative.

However, the truss alternative requires the use of K-nodes, which are definitely more complicated to fabricate than T-nodes and certainly more costly. In addition, the stress concentrations around K-nodes are slightly higher than for T-nodes. Hence, the choices between both alternatives of truss and frame arrangement are mostly depending on the aesthetic value of the bridge.

Since the nodes of tubular arch bridges are equally important as is the brace arrangement, and in order to reduce stress concentrations, an optimal location of internal diaphragm has been derived. This location may vary if fatigue resistance proves to be an important issue for the bridge.

Three scaled tests, applying torsion to a steel tube, encased on a sufficiently large concrete slab is of course a limited number to draw general conclusions. The number of tests should be considerably larger and some characteristics of the test including headed studs should be modified. In addition, the combination of various types of connectors should also be considered. Nevertheless, the 3 tests have demonstrated or confirmed some properties of strip and stud connectors, subjected to torsion.

Natural bond does not contribute to the connection of steel and concrete, as shown by the result of the first test, which obviously also served for improving the setup and equipment.

The second test has demonstrated that from the first loading a composite action of steel and concrete must exist. Three successive mechanisms for resistance of this type of connector were found. In addition, the strip connection is rather flexible for torsion and does not really act as perfect clamping, since it appeared to be 70-times weaker than the torsion stiffness of the steel tube.

The third test showed that headed studs need to be supplemented by rebars, since failure occurred prematurely, due to concrete splitting, starting at the contact area of the stud shaft. This is easily avoided by normal reinforcement. The torsion stiffness of the connection is distinctly higher compared to the previous type.

In relation to the excessive lateral displacement of the arches of the Moerbrug bridge, during concrete casting, an attempt was made to relate it to torsional stiffness of the arch spring connection. The result indicates that the studs inside the arch closed section are probably ineffective. Any torsion stiffness must be provided by the strips. However, scaling of the test results demonstrates that the arch spring almost behaves as a free torsion condition.

Further experimental research would be welcomed to improve knowledge about this torsion connection and might perhaps result in establishing more reliable data about the torsion stiffness of steel bridge superstructure in concrete abutments.

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SUMMARY

BRACE ARRANGEMENT, NODE STIFFENING AND CONNECTION OF SPRINGS OF STEEL TUBULAR ARCH BRIDGES ***MIHA TOMAŽEVIČ***

Em Philippe VAN BOGAER

Three characteristics of steel tubular arch bridges are being discussed. The brace members, connecting the main chord tube to an upper concrete or composite deck can be arranged either as triangular truss members, or as rigid node vertical or slightly sloping frame members. As can be expected, the former arrangement results in lower bending of the arch chord and lower compression near its centre. However, the number of brace tubes for triangular arrangement is twice as much as for the frame system. In addition, the nodes of the truss system necessarily join at least 4 tubes and are of the K-type. The latter are more complicated to design and to fabricate than T-nodes, used in the frame arrangement.

Fatigue resistance of welded tubular nodes is rather low. Should the chord diameter be sufficiently large, internal diaphragm stiffeners can be provided, thus increasing considerably the fatigue resistance. Both numerical and experimental research demonstrates that an optimal location of the diaphragms can be found. However, this optimum location does not necessarily correspond to the minimum of stress concentrations.

Most springs of steel tubular arch bridges are rigidly connected to a concrete abutment. Generally pre-stressing bars are used to achieve this. Unfortunate experience has shown that this connection may still be too flexible to prevent axial rotation due to torsion. The alternative of using various connectors has been implemented in few bridges. The torsion resistance of such a connection has been assessed in 3 scaled tests. The results have given more insight into the mechanism of torsion connection and results in experimental values of the torsion stiffness. The preliminary conclusion of this research indicates that stud connectors show larger stiffness than strips, which is a rather unexpected outcome.

Keywords: steel tubular arch bridge, fatigue resistance, arch spring torsion, strip stud connectors, tubular node stiffening, rotation stiffness

APSTRAKT

RASPORED UKRUĆENJA, UKRUĆENJE ČVOROVA I VEZA S KRAJNJIM OSLONCIMA ČELIČNIH CEVASTIH LUČNIH MOSTOVA

Em Philippe VAN BOGAER

Razmatraju se tri karakteristike cevastih lučnih mostova. Elementi ukrućenja, koji povezuju glavnu pojasnu cev s gornjom betonskom ili spregnutom pločom mogu biti postavljeni ili kao trougaone rešetke, ili kao ram s krutim čvorovima i vertikalnim ili blago nagnutim elementima. Kao što se može očekivati, prva postavka daje manje savijanje pojasa luka i manji pritisak blizu njegove sredine. Međutim, broj cevastih ukrućenja za trougaonu postavku dvostruko je veći u odnosu na ramovski sistem. Pored toga, čvorovi rešetkastog sistema nužno povezuju bar četiri cevi i oni su tipa „K”. Ovo potonje je kompikovanije za projektovanje i izvođenje od T-čvorova korišćenih u ramovskoj postavci.

Otpornost na zamor čvorova zavarenih cevi veoma je niska. Ako bi prečnik pojasa bio dovoljno velik, unutrašnja ukrućenja u vidu dijafragmi mogu se ugraditi i tako značajno povećati otpornost na zamor.

I numerička i eksperimentalna istraživanja pokazuju da je moguće naći optimalni položaj dijafragmi. Međutim, ovaj optimalni položaj ne mora nužno odgovarati minimalnim koncentracijama napona.

Većina krajnjih oslonaca cevastih lučnih mostova kruto je vezana za betonske oslonce. Generalno se koriste prednapregnute šipke da bi se ovo postiglo. Negativna iskustva su pokazala da ova veza ipak može biti previše fleksibilna da bi sprečila aksijalnu rotaciju usled torzije. Alternativa tome bila je upotreba različitih konektora koja je primenjena kod nekoliko mostova. Otpornost na torziju ovakvih konektora ocenjivana je putem tri opita u smanjenoj razmeri. Rezultati su pružili više uvida u mehanizam torzione veze i dobile su se eksperimentalne vrednosti torzione krutosti. Preliminarni zaključak ovog istraživanja ukazuje na to da konektori s moždanicima iskazuju veću krutost nego trake, što predstavlja prilično neočekivan ishod.

Ključne reči: čelični cevni lučni most, otpornost na zamor, torzija oslonca luka, konektori s trakastim moždanicima, ukrućenje cevnog čvora, rotaciona krutost

NONLINEAR SEISMIC ANALYSIS OF MASONRY INFILLED RC FRAME STRUCTURES

NELINEARNA SEIZMIČKA ANALIZA AB OKVIRNIH KONSTRUKCIJA SA ZIDNOM ISPUNOM

Emin MAHMUD
Zdravko BONEV
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PRETHODNO SAOPŠTENJE
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1 INTRODUCTION

Considering recent earthquakes happened during the last decade the seismic performance of reinforced concrete frames with masonry infill is a subject of great interest. Earthquakes in Chile (2010), Haiti (2010), New Zealand (2010) and Japan (2011 and 2018) can be quoted as examples providing about what is needed to be done in the future [1].

The behaviour of frames with masonry infill in seismic actions is associated with many specifics:1) both materials (masonry infill and reinforced concrete) have different deformation characteristics ductility ratios; 2) considering earthquake loading it is observed that masonry stiffness is slowly going down showing the tendency of growing up the cracks and damages. This process is found to be with reference to earthquake duration and earthquake intensity; 3) Compliance between masonry infill walls and reinforced concrete frame is generally dependent on interface surface considered as contact zones between both materials; 4) It is the purpose of the paper to carry out a proper investigation and develop a procedure for fast evaluationof masonry infills influence on seismic demand and on overall seismic performance of the structure

The overall model of a framed masonry structure consists of two substructures – reinforced concrete frame and masonry (as diagonal element) introduced by its specific parameters using an appropriate procedure based on laboratory data. Inelastic deformation of both substructures implies that overall behaviour of framed masonry is specified as nonlinear and structure should be studied on the basis on nonlinear analysis.

In this paper an analysis has been carried out to evaluate the influence of masonry infill on the seismic performance of one day single storey plane reinforced concrete frame structure. Three models for numerical analyses are performed considering the reinforced concrete frame. At first, response spectrum method is applied to determine the design of seismic load and corresponding action effects according to the National code and European provisions. This model is denoted as model M1. After determination of design seismic loads, the design action effects are determined through linear and response spectrum analyses. After this step columns and girder of the frame are designed in conformity with European standards and European detailing rules. In the next second model M2 the contribution of masonry infill is not taken into consideration because masonry infill is not in use. This model is capable to develop inelastic deformations and should be studied based on nonlinear analysis. The third model M3 contains masonry infill and its contribution to the frame stiffness is considered using equivalent diagonal strut element (corner to corner model). To model the strut diagonal in *SAP2000 v20* [2] multi-linear plastic link elements are used.

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2 EQUIVALENT DIAGONAL COMPRESSION STRUT

Aiming to overcome the complexity of the numerical models and optimize the procedures known as much time consuming it is a good idea to use macro elements to model masonry infill. The most elementary one is the diagonal strut element which is capable of simulating the infill wall resistance to lateral seismic forces. The basic idea of using strut models is to determine the global effect of masonry infill on the global behaviour of RC frame – infill structure. Analytical and experimental studies carried out by Polyakov [3], [4] show properly formation of compression diagonal strut. The basic problem is to identify physical properties of this element.

At first the concept is developed assuming only one diagonal strut connecting upper and lower edges of the masonry panel by following compression diagonal geometry. In order to generalize the model including cyclic behaviour strut elements are placed in each diagonal direction of masonry panel. Having in mind that tensile stress is small enough compared to compression ones it is reasonable to accept that the level of tensile stress is negligibly small and can be assumed to be zero. This means that only compression diagonal element resists to horizontal seismic force. Two varieties for diagonal struts can be used – concentric and eccentric [1], [5], [6] see Fig. 1.

During the last two decades it becomes clear that the model containing the only one element is incapable to reproduce the complex seismic performance of masonry infill. A number of researchers (Chrysostomou et el. [7], [8], Crisafulli [9] El-Dakhkhni et al. [10], [11], [12], Crisafulli and Carr [13], Rodrigues et al. [14], [15], Mihaleva [6] et al.) modified firstly proposed single diagonal using few diagonals gathered in one model. As a result of this the contact zones and interface surfaces are more precisely modelled especially opening the tensile surfaces located at the edge. Thus, new complex element accounts for larger tensile released stress zone. It is capable of modelling shear strength greater than the corresponding shear strength of surrounding framing elements. It is worth to mention the conclusion that the complex element is more accurate considering the edge contact zones, and the greatest advantage of a single diagonal model is its simplicity.

A number of researchers propose various relationships to determine the equivalent width of the strut compression diagonal. According to Holmes [16] the width of the equivalent compression diagonal (b_{wiinf}) is 33% of the diagonal length of the infill panel. Paulay and Priestley [17] recommended the equivalent strut width to be specified as 25% of the panel diagonal. They concluded that the bandwidth does not depend on the lateral stiffness of the framing system. Abdelkareem, Cathrin et el. [18], [19] considered various approaches for determination of b_{winf}. The authors concluded that the strut width ranges between 1/10 to 1/3 of the geometric length of masonry panel [6], [18].

There are not exact provisions in Eurocode 8 [20] about the equivalent compression strut width. In FEMA-306 [21] b_{winf} the model of masonry infill proposed is expressed by the following quantities:

$$\lambda_1 = \sqrt[4]{\frac{E_{winf} t_{winf} \sin(2.\theta)}{4.E_{cm} I_c h_{winf}}} \tag{1}$$

$$b_{winf} = \frac{0{,}175 r_{winf}}{(\lambda_1.l_c)^{0{,}4}} \tag{2}$$

where:

- b_{winf} – equivalent diagonal strut;
- r_{winf} – diagonal length of infill panel;
- θ – angle, whose tangent is the infill height-to-length aspect ratio;
- h_{winf} – height of infill panel;
- t_{winf} – thickness of infill panel and equivalent strut;
- E_{winf} – modulus of elasticity of infill material;
- λ_1 – function of the relative panel-to-frame-stiffness parameter;
- l_c – column height between centrelines of beams;
- I_c – moment of inertia of column;
- E_{cm} – modulus of elasticity of frame material

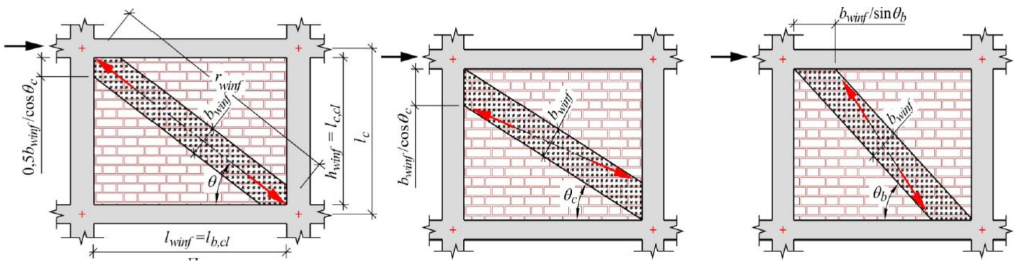

Fig. 1. Strip defined diagonal containing a single equivalent strut element [5]

3 SHEAR CAPACITY OF MASONRY INFILL

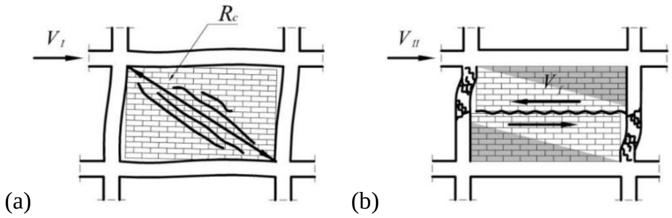

Fig. 2. Most frequently considered failure modes of masonry infill: failure of compression diagonal (a) and horizontal sliding of masonry panel (b) [22]

Mainly two failure modes of masonry infill panel are dominant when structures are subjected to seismic action. First one is characterized by failure of compression diagonal (Fig. 2a). Second failure mode is related to horizontal shear (sliding mode) of the masonry panel (Fig. 2b) [22]. Horizontal capacity of the masonry panel is determined on the basis of horizontal resistance of masonry panel, based on horizontal capacity of the horizontal joints $F_{winf,hor,sf}$, which is accepted to be equal to initial strength of masonry in shear assuming zero compression strength f_{v0d}. For masonry infill with thickness t_{winf} and length l_{winf} horizontal capacity of the joints is determined using expression (3). Equivalent diagonal compression capacity is determined using formula (4), as well,

$$F_{winf,hor,sf} = f_{v0d} t_{winf} l_{winf} \tag{3}$$

$$F_{winf,hor,cf} = f_d t_{winf} b_{winf} . \cos\theta \tag{4}$$

where:

- $F_{winf,hor}$ – horizontal capacity of the masonry panel
- f_{vod} – design initial strength in shear of masonry assuming zero compressive stress;
- f_d – design strength of masonry;
- b_{winf} – equivalent diagonal strut;
- t_{winf} – thickness of infill panel and equivalent strut;
- l_{winf} – length of infill panel;
- θ – angle, whose tangent is the infill height-to-length aspect ratio;

4 ANALYSIS OF FRAMES CONTAINING INFILL MASONRY

Single storey one bay reinforced concrete frame with brick masonry panel is studied. Cross section dimensions are given as follows: columns 400/400 mm, beam 300/400 mm and thickness of masonry infill – 250 mm (see Fig. 3, Fig. 4, Tab. 1 and Tab. 2). The frame is loaded using distributed trapezoidal load. Permanent uniformly distributed loads including all loads of this type is g_k=45kN/m', service loads are assumed q_k=15kN/m'.

The seismic design of the frame is carried out aiming the frame to withstand the design earthquake for seismic zone Blagoevgrad (Bulgaria), characterized by reference ground acceleration for free field a_{gR}=0,32.g, importance factor γ_I=1,00 и coefficient of behaviour q=2,00 (it is assumed that the forces as action effects coming from the design earthquake, are reliable). Ground type B and response spectrum of Type 1 according to the National code [20] are also used. In addition, nationally determined parameters from the Bulgarian National Annex [23] should be mentioned as important for design seismic action definition.

To reach the paper purposes three static analyses are performed. First analysis is linear, the other two analyses are nonlinear. In first nonlinear analysis masonry infill is not accounted for. In the next nonlinear analysis, the masonry infill is taken into considerations. Seismic analyses are performed using *SAP2000 v20* computer software [2]. Three different numerical models are applied. Seismic action is assumed as single component (horizontal) and located within the frame plane.

4.1 Numerical model M1

The numerical model M1 is used to carry out the first step of the analysis scheduled – seismic design procedure. To do this response spectrum analysis is applied in combination with linear analysis. At this stage masonry infill is not accounted for. The output of this step is that the structure is designed and detailed according to Eurocode 1998-1 [20], [23] to withstand the design earthquake. The reinforcement obtained as a result of the design procedure application is provided in Table 1:

Tab. 1. The quantity of reinforcement used in columns and beam

member	Concrete	Reinforcing steel	Cross section	Longitudinal reinforcement	Transverse reinforcement	
beam	C25/30	B500C	300 x 400 mm	4∅16 (top) and 4∅14 (bottom)	∅8/100	∅8/200
columns	C25/30	B500B	400 x 400 mm	8∅16	∅8/125	∅8/250

4.2 Numerical model M2

In Numerical model M2 a non-linear static analysis is performed. This model fails to include masonry infill into the calculations. Analysis is carried out assuming permanent vertical loading and lateral load increasing stepwise. Columns and girder are connected using rigid rotational links in their joints. Plastic deformations are considered by implementation of plastic hinges into the numerical model M2 close to beam-to-column joints, see Fig. 3.

Specific data for plastic hinges is prepared including the reinforcement available. Plastic hinges are modelled by using inelastic rotation spring interconnecting two neighbouring cross sections. Constitutive material relationship for rotational spring of the type "bending moment – axial force – relative rotation of the cross sections" is used. Both cross sections are interconnected with the rotational spring. The bending moment in rotational spring is dependent on the axial force more essentially in the columns, *(M_3-P).*

4.3 Numerical model M3

To define inelastic cyclic behaviour of masonry, infill the *Hysteretic Pivot model* [2] (Fig. 5a.) is used. An inelastic static analysis is performed over the RC frame using the formulation M3, see Fig. 4. The proposed equivalent strut model is employed and illustrated in Fig. 4. Models M2 (see Fig. 3) and model M3 are similar and convenient to be numerically compared, but the only difference is that M3 includes the contribution of the masonry infill through the strut diagonal member. In *SAP2000 v20* [2] the equivalent strut diagonal is modelled by *multi-linear plastic link element.* This model has large flexibility in modelling structures developing unsymmetrical hysteretic loops under the action of axial force in compression-tension. Hysteretic loops are numerically controlled through the parameters α_1, α_2, β_1 *u* β_2. The *Hysteretic Pivot model* has specific and simplified rules when applied to tensile equivalent strut members (Fig. 5b.).

Hysteretic rules are load dependent and numerically controlled using the parameters α_1, α_2, β_1 *u* β_2. The use of *Hysteretic Pivot model* has the advantage to offer simplified rules to be applied to equivalent diagonal strut (Fig. 5b.). In this case, since the tensile strength of masonry infill is not considered, the parameters α_1 and β_1 are generated both to be zero [24]. The hysteretic model specified herein has another advantage – generates always positive stiffness. In dynamic analysis the use of negative or zero tangential stiffness may influence the stability of the numerical solution.

Experimental investigations show that for frames containing masonry infill panels the change of the sign of seismic action does not lead to increase of the stiffness. It is established that the β_2 is also zero. Hysteretic loop controlled its area through the parameter α_2. The value of 0.25 is taken the following reference [24].

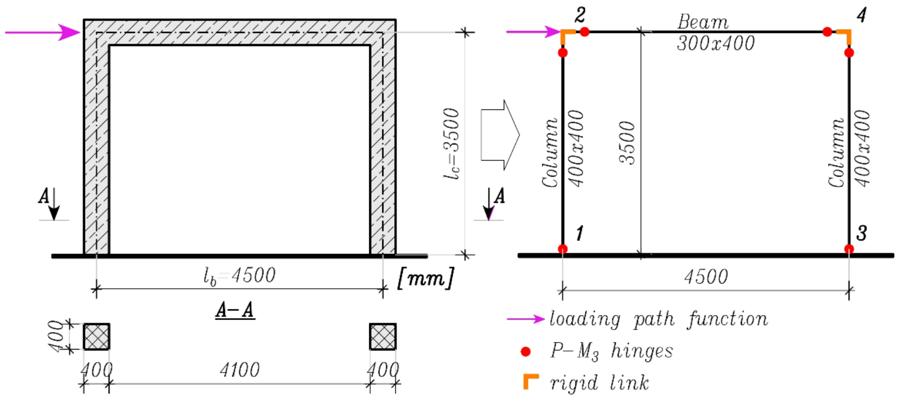

Fig. 3. Structure model based on formulation M2

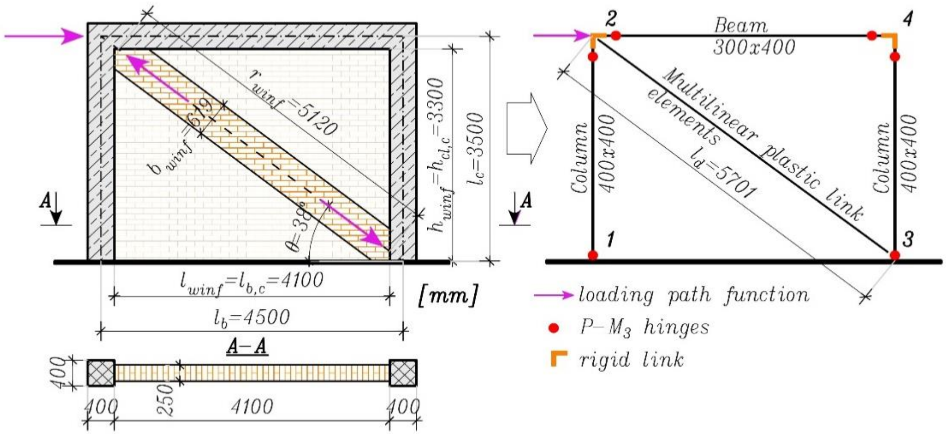

Fig. 4. Structure model based on formulation M3

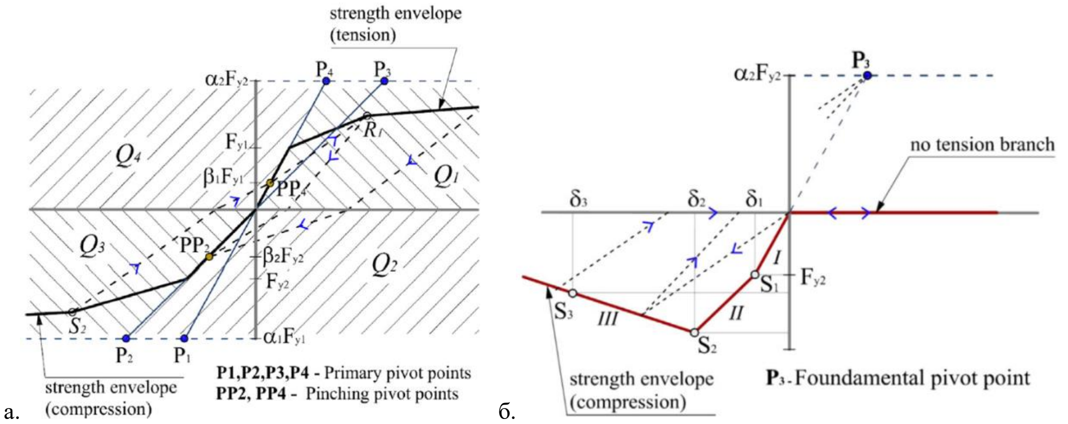

Fig. 5. Hysteretic Pivot model: a) unsymmetrical hysteretic loop and b) equivalent compressive diagonal [24].

Due to define and compose equivalent diagonal strut model, characteristics of masonry are determined in conformity with European standard Eurocode 1996-1-1 [25], [26] and Eurocode 1998-1 [20] for seismic resistant building structures. A block structure for masonry infill of Group 2 is used and mortar containing lightweight additive materials having volume weight *$\rho_d \leq 800kg/m^3$*. Characteristic strength of masonry f_k is

$$f_k = K f_b^{0,70} f_m^{0,30} = 0,25.10^{0,70}.5^{0,30} = 2,03\ MPa \tag{5}$$

The design strength of masonry f_d with blocks of category I and prescribed dimensions are given as:

$$f_d = \frac{f_k}{\gamma_M} = \frac{2,03}{2,00} = 1,01\ MPa \tag{6}$$

The characteristic initial strength in shear of masonry assuming zero compressive stress and made of clay blocks and mortar with light additive materials is *$f_{vok}=0,15MPa$* and the design strength is *$f_{vod}=0,075MPa$*.

The durable secant modulus of masonry is:

$$E_{long\ term} = \frac{1000.f_k}{1+\phi_\infty} = \frac{1000.2,03}{1+1,10} = 967\ MPa \tag{7}$$

The width within the model of equivalent diagonal strut in compression is derived after using equations (1) and (2):

$$\begin{aligned}\lambda_1 &= \sqrt[4]{\frac{E_{winf} t_{winf} \sin(2.\theta)}{4.E_{cm} I_c h_{winf}}} = \\ &= \sqrt[4]{\frac{967.250.\sin 76}{4.31000.\frac{400.400^3}{12}.3300}} = 7,1992.10^{-4} mm^{-1}\end{aligned} \tag{8}$$

$$b_{winf} = \frac{0,175 r_{winf}}{(\lambda_1.l_c)^{0,4}} = \frac{0,175.5120}{\left(7,1992.10^{-4}.3500\right)^{0,4}} = 619mm \tag{9}$$

The minimum shear capacity basing on both possible failure mechanisms, is:

$$\begin{aligned}V_m &= F_{winf,hor} = \min\begin{cases}F_{winf,hor,sf}\\ F_{winf,hor,cf}\end{cases} = \\ &= \min\begin{cases}0,075.250.4100\\ 1,015.250.619.\cos 38^\circ\end{cases} = \min\begin{cases}76,9\\ 123,8\end{cases} = 76,9\ kN\end{aligned} \tag{10}$$

Tab. 2. Parameters defining Hysteretic Pivot model

l_{winf}	t_{winf}	θ	f_{v0d}	f_d	l_d	b_{winf}	$F_{winf,hor,cf}$
mm	mm	o	MPa	MPa	mm	mm	kN
4100	250	38	0,075	1,02	5701	619	123,80

$F_{winf,hor,sf}$	V_y	V_m	V_p	δ_1	δ_2	δ_3
kN	kN	kN	kN	mm	mm	mm
76,90	57,7	76,9	15,4	-10,9	-14,5	-25,3

The point S2 in *Hysteretic Pivot model* for equivalent diagonal replacing strut (Fig. 5b.) is defined when both maximum capacity of masonry panel V_m and displacement δ_2 are reached. The point S1 is defined when 75% of V_m (V_y=0,75.V_m) is reached and displacement becomes δ_1**.** PointS3 is defined when the peak of the strength drop to 20% of V_m(V_p=0,20.V_m) is reached and displacement becomes δ_3.

The displacements δ_1, δ_2, δ_3 are determined based on normal strains ε_i pointed out in [27]. Table 2 contains defining parameters of *Hysteretic Pivot model*.

5 ANALYSIS OF THE RESULTS ON OUTPUT

Inelastic static performance of framed masonry is presented by means of the curve "base shear force – roof horizontal displacement". Numerical models M2 and M3 are used to evaluate the contribution of masonry infill on the seismic demands for roof displacements. The results can be seen in Fig. 6 which represents in graphical format the curves "base shear force – horizontal displacement at the top". Both curves are obtained computationally using computer software. The curves are also called "capacity curves". Capacity curves for structure without masonry infill are denoted as violet lines, whereas the same curves for structure with masonry infill are denoted with red dashed lines.

In general, the horizontal base shear force in case of added diagonal strut is formed as a sum of the frame base shear and horizontal projection of the diagonal force in compression. External loading is shown in Fig. 6 coloured in pink. The resistance force has two counterparts – first is coming from the reinforced concrete frame, second appears to be a result of added masonry infill represented in the computer model as diagonal strut. Both forces are internal forces, see Fig. 6. Internal forces must equilibrate the external forces, see Fig. 6. Thus, the frame resists to progressively increasing external static loads through internal resisting forces. Furthermore, internal forces depend on displacements and this relationship is as a rule implicit and nonlinear. That is why the solution of the nonlinear static problem is found using iterations. After each iteration internal forces need to be revaluated and rest between external and internal forces is getting smaller. If external and internal forces become equal, equilibrium is reached. In the descending branch of the capacity curve internal force is going down. This means that internal force is no capable of resisting to increasing external forces any further. If equilibrium cannot happen any longer, the process of failure occurs. The initial stage of loading history is characterized with the fact that the system behaves itself linearly. After the development of inelastic deformations, the capacity curve properly expressed nonlinear behaviour (Fig. 6). Masonry infill contributes towards reduction of displacement demand. On the other hand the capacity in lateral direction is increased. When further stages of loading history are considered, it is observed, that the masonry infill is participated more actively (Fig. 6), in formation of the lateral resistance. Having this in mind, the maximum shear force at the base is reached the amount of 191,50 kN as observed (point E).

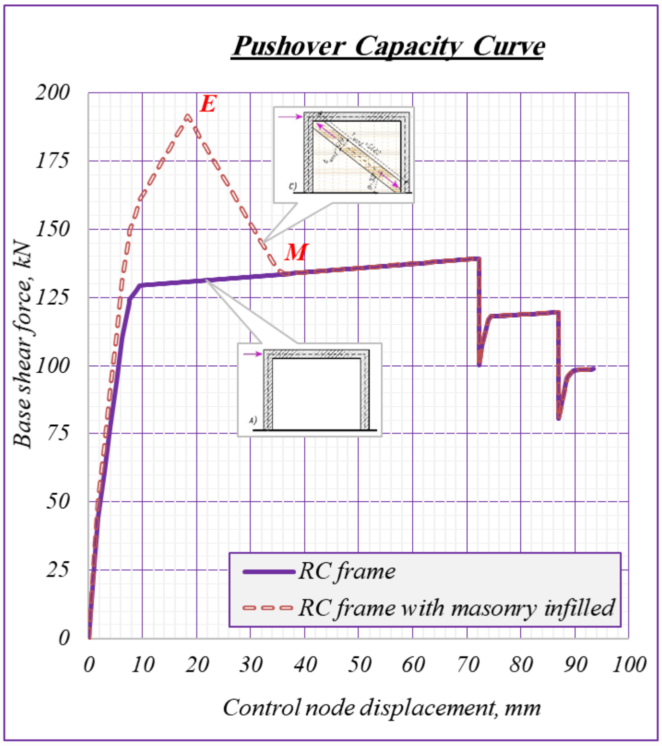

Fig. 6. Capacity curves of reinforced concrete frame (a) with (red dotted line) and without (violet line) masonry infill

After reaching point E, intensive crack initiation, capacity degradation and stiffness of the masonry are noticed. Its resistance essentially drops down. When base shear force is 133,60kN (at about 70% of shear capacity), masonry infill is incapable of resisting to horizontal loads anymore. Point M (Fig. 6) is intersection point for two capacity curves. At point M the masonry infill is fractured and damaged. For displacement demands greater than 0.036m both capacity curves (frame with masonry infill and frame without infill) completely fit each other. Dissipated seismic energy for frame with masonry infill is 23% greater than dissipated energy for frame without masonry infill.

In references [20], [28] the procedure of evaluation of design seismic performance is considered and discussed. The N2-method is focused on determination of the performance point. It is found as intersection point between capacity spectrum and design demand spectrum (Fig. 7). The comparison between calculated seismic displacement demand S_d and maximum admissible level of structural performance is a good opportunity to see whether the structure is in conformity with new generation of code provisions or not.

Fig. 7 illustrates the application of N2 method [29] to find out the displacement seismic demand. The design seismic action is presented using *"design demand spectrum"* plotted in S_a-S_d format (acceleration-displacement format). Spectral acceleration can physically be treated as a *"force per unit mass"*. Thus, the spectral acceleration S_a-can be identified as external quasi-static force used to load the structure in compliance with the design seismic action. The capacity curve provides the resistance spectrum and the equality between external and internal forces means that both forces are in equilibrium. Obviously, this happen when displacement has the demand value called also "target displacement". It is evident from Fig. 7 that at intersection points P_1 and P_2 structural performance is ductile and collapse is not reached.

The influence of masonry infill on design seismic performance of the frame is illustrated in Fig. 6 and Fig. 7. The evaluation is based on comparison of the position of performance point of both capacity spectra – frame with masonry infill (red dotted line) and without infill (violet line). In the first case (red dotted line) design displacement demand is 2,3 times smaller than the result of violet line, where no masonry infill is considered.

6 CONCLUSIONS

Nonlinear static analysis of a RC frame is performed. Two well known cases from the practice are studied - the frame with masonry infill (case 1) and the frame without masonry infill (case 2). The analysis is carried out assuming constant vertical load and horizontal load increasing stepwise. As a result, two capacity curves are presented (for case 1 and case 2). Design seismic demands are evaluated for both cases. The benefit of using masonry infill is clarified and explained.

Masonry infill on one hand increases static lateral stiffness of overall structure and resists netter to seismic loads, on the other hand base shear capacity is increased. After reaching the greatest horizontal static level of loading the process of intensive cracking is observed, stiffness degradation occurs and capacity steeper goes down. The structure has no capability to resist external horizontal loads.

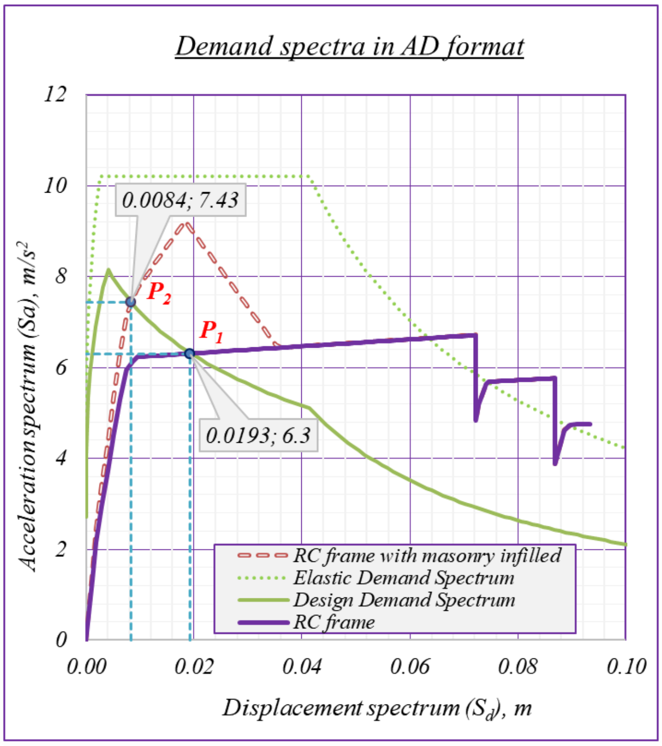

Fig. 7. Determination of performance point and displacement demands for structure without masonry and with masonry

At the beginning of the loading process the linear behaviour is dominant in the performance. Initial stiffness of the frame containing masonry infill K_0 is 20% greater compared to initial stiffness $K_{0,f}$ of the frame without masonry infill. After reaching limit value of the horizontal load starts intensive cracking and stiffness degradation. As a result of this the capacity of masonry infill in carrying horizontal loads is drastically decreased. Looking at capacity curves it can be properly seen that after ending this process contribution of masonry disappears and both curves are the same.

Dissipated seismic energy of the frame is increased 23% due to masonry infill considering frame masonry. It is numerically proven in the present paper that the main benefit of using masonry infill in combination with reinforced concrete frame is the displacement seismic demand reduction and corresponding increased capacity of horizontal seismic resistance. Due to increase the dissipated seismic energy, it is necessary to improve the property of masonry, which is non-ductile material. This improvement can be achieved by reinforced masonry, using friction dampers and others.

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ABSTRACT

NONLINEAR SEISMIC ANALYSIS OF MASONRY INFILLED RC FRAME STRUCTURES

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Emad ABDULAHAD

Structures composed of different substructures and materials are a subject of great research and expert interest. It is the purpose of this research to develop an engineering approach for evaluation of seismic demands and to apply the methodology to reinforced concrete frames with or without masonry infill. This paper deals with combination of reinforced concrete frame and masonry infill wall. It is known that the combined structure resists well to severe earthquakes. In this paper, the influence of masonry infill on the seismic demands of a reinforced concrete frame structure is estimated using nonlinear analysis. A strut model was used to model the masonry infill resistance; the strut model is statically equivalent to the masonry infill wall. Therefore, material properties of the strut element are determined based on material properties of continuous masonry wall and geometry data.

The seismic analysis is performed including material nonlinearity of the model. Determination of seismic displacement demands is carried out using an approximate procedure of the nonlinear analysis in N2-method. To make evaluation of masonry contribution two varieties of the models have been studied – “with masonry infill” and “without masonry infill”.

Key words: Masonry infill, nonlinear analysis, seismic action, seismic behaviour, N2-method, RC frame structure, response spectrum method

REZIME

NELINEARNA SEIZMIČKA ANALIZA AB OKVIRNIH KONSTRUKCIJA SA ZIDNOM ISPUNOM

Emin MAHMUD
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Emad ABDULAHAD

Konstrukcije sastavljene od različitih potkonstrukcija i materijala jesu tema brojnih istraživača i interesa stručnjaka. Svrha ovog rada jeste da se razvije inženjerski pristup za procenu seizmičkog odgovora i da se metodologija primeni na armiranobetonske (AB) okvire sa ispunom i bez ispune. Ovaj rad bavi se kombinacijom AB okvira i zidane ispune. Poznato je da su kombinovane konstrukcije otporne na jake zemljotrese. U ovom radu uticaj zidane ispune na seizmički odgovor AB okvira rešava se nelinearnom analizom. Metoda pritisnutog štapa primenjena je da bi se modelirao otpor zidane ispune, a metod pritisnutog štapa je statički ekvivalentan zidanoj ispuni. Dakle, materijalna svojstva pritisnutog štapa određena su na osnovu svojstava materijala zidane ispune i geometrijskih podataka.

Seizmička analiza sprovedena je uzimajući u obzir materijalnu nelinearnost modela. Određivanje pomeranja − kao seizmičkog odgovora − sprovedeno je približnom metodom nelinearne analize N2. Prednost ispune u seizmičkom odgovoru ispoljava se smanjenjem pomeranja. Da bi se napravila procena uticaja ispune, analizirane su dve varijante modela − *sa zidanom ispunom* i *bez zidane ispune*. U prvom slučaju, sračunat je uticaj zidane ispune. U drugom slučaju, otpor zidane ispune nije uziman u obzir i nije uključena „pomoć” zidarije.

Ključne reči: zidana ispuna, nelinearna analiza, seizmičko dejstvo, seizmičko ponašanje, N2 metoda, AB okvirne konstrukcije, metod spektra odgovora

POREĐENJE PONAŠANJA TANKIH CILINDRIČNIH I KONUSNIH LJUSKI OD UGLJENIČNOG I NERĐAJUĆEG ČELIKA

BEHAVIOUR OF THIN-WALLED CYLINDRICAL AND CONICAL SHELLS - CARBON vs. STAINLESS STEEL

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PRETHODNO SAOPŠTENJE
PRELIMINARY REPORT
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1 UVOD

Tanke cilindrične ljuske su konstruktivni elementi koji su našli široku primenu u praksi. Bilo da je reč o limenci piva ili o delu za rakete, neophodno je temeljno poznavanje svih njihovih karakteristika. Posebno treba naglasiti analizu stabilnosti i raspored (formu) izbočine koja se formira u postkritičnoj oblasti − dijamantsku formu, ili u literaturi još poznatu kao Yoshimura šablon [19].

Stvarne ljuske su elementi koji imaju određene imperfekcije. Moguće je klasifikovati imperfekcije na tri osnovna tipa: geometrijske, strukturne i imperfekcije u opterećenju. U analizi, najčešće se koriste geometrijske imperfekcije zbog jednostavnosti njihovog definisanja, a pokazale su se i kao adekvatne za opisivanje bilo koje vrste imperfekcija. Dobro je poznato da najveći uticaj na stabilnost cilindra imaju geometrijske imperfekcije zadate u obliku koji odgovara sopstvenim oblicima izbočavanja (jednom imperfekcijom ili kombinacijom više njih). Ovakva pretpostavka značajno odstupa od realne slike, ali je u građevinarstvu našla široku primenu. Jedan od razloga za to jeste mali broj dostupnih merenja imperfekcija na

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1 INTRODUCTION

Thin-walled cylindrical shells are structural elements that are widely used in practice. No matter if it is about a beer can or a rocket section, it is necessary to thoroughly know all their characteristics. The analysis of stability and buckling pattern formed in the post-buckling area should be especially emphasized. Buckling pattern can be a diamante pattern or the Yoshimura pattern, as it alternative known in literature [19].

The actual shell structures are elements which have certain imperfections. It is possible to classify these imperfections to three basic types: geometrical, structural and loading imperfections. The most often used are geometrical imperfections due to the simplicity in their definition. Moreover, it is proved that geometrical imperfections are adequate for describing any kind of imperfections. It is well known that geometrical imperfections have the highest influence on the cylinder stability. The geometrical imperfections are set in the form which corresponds to the eigenmodes (one or several of them combined). Such assumption considerably deviates from

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izvedenim konstrukcijama. Pored velikih ekonomskih izdataka, koji su neophodni da bi se jedno ovakvo merenje izvršilo, treba napomenuti i to da su imperfekcije funkcija s velikim brojem promenljivih. Znatan uticaj na pojavu imperfekcija imaju različite faze gradnje, način eksploatacije, klimatski uslovi (temperatura), kao i mnogi drugi uslovi.

U građevinarstvu, izbočavanje cilindrične ljuske merodavan je kriterijum za dimenzionisanje dimnjaka, stubova vetrogeneratora, silosa i sličnih objekata. Za praktičnu primenu, potrebno je pronaći ravnotežu između sigurnosti i ekonomičnosti, zadržati se u finansijski opravdanim granicama, primenom manje konzervativnih rešenja, ali ostati na strani sigurnosti u pogledu nosivosti i stabilnosti. U svemu navedenom leži i cilj istraživanja ovog rada, a to jeste uticaj različitih vrednosti amplituda početnih geometrijskih imperfekcija, koje su zadate u obliku prvog sopstvenog oblika izbočavanja, na ponašanje ljuski, uz analizu izbora materijala od kog su ljuske izrađene.

U radu su prikazani rezultati numeričke analize stabilnosti cilindričnih i konusnih ljuski, primenom programa Abaqus. Analiziran je uticaj materijalne i geometrijske nelinearnosti na kritičan napon i nosivost na izbočavanje. Materijalna nelineranost uzeta je u proračun primenom eksperimentalnih krivih napon–dilatacija ugljeničnog i nerđajućeg čelika, dok je geometrijska nelineranost obuhvaćena različitim vrednostima početnih imperfekcija. Rezultati sprovedene numeričke analize poređeni su s preporukama datim u EN 1993-1-6 [9] i definisane su vrednosti redukcionog faktora kao odnosa kritičnog napona i nosivosti na izbočavanje. Vrednosti redukcionog faktora poređene su s podacima dostupnim u referentnoj literaturi.

2 AKSIJALNO OPTEREĆENE KRUŽNE CILINDRIČNE I KONUSNE LJUSKE

Tokom poslednjih decenija, veliki broj istraživanja bio je usmeren na analizu problema stabilnosti ljuski. Naročito se izdvojio slučaj aksijalno napregnutih cilindričnih ljuski izloženih dejstvu pritiska, kao karakterističan za ovaj tip konstruktivnih elemenata, koji je u samom radu i analiziran. Razlog za to jeste značajno drugačije ponašanje cilindričnih ljuski pri aksijalnom opterećenju u poređenju s ponašanjem ploča i stubova [3]. Navedeno je ilustrativno prikazano na slici 1 za različite konstruktivne elemenate izložene dejstvu aksijalnog pritiska. Prikazane su krive sila–deformacija dobijene analizom ponašanja za inicijalnu fazu, graničnu fazu pri kojoj dolazi do gubitka stabilnosti i postkritičnu fazu. Za slučaj elemenata bez početnih imperfekcija, na slici 1 uočava se da se nakon dostizanja Eulerovog kritičnog opterećenja u slučaju ploča opterećenje (*P*) povećava s povećanjem aksijalne deformacije (Δ). Kod stubova ono se ne povećava, ali se i ne smanjuje, dok kod cilindričnih ljuski s daljim porastom deformacija (Δ), opterećenje (*P*) opada nakon dostizanja Eulerovog opterećenja. Kada se uzme u obzir uticaj početnih imperfekcija, kod stubova se može uočiti da je Eulerovo opterećenje maksimalna nosivost za sve aksijalno pritisnute elemente, s početnim imperfekcijama i bez njih. Kod ploča se uočava porast deformacije (Δ) u slučaju elementa s početnom imperfekcijom, ali oba slučaja mogu da prihvate opterećenje

the actual condition, but it has been widely used in civil engineering. One of the reasons is the small number of available imperfection measurements on the structures. In addition to the considerable financial cost that is necessary to perform such a measurement, it should be highlighted that imperfections are the function with a large number of variables. Different phases of construction, and the way of operation, climate conditions (temperature) and many other factors have a great impact on imperfections.

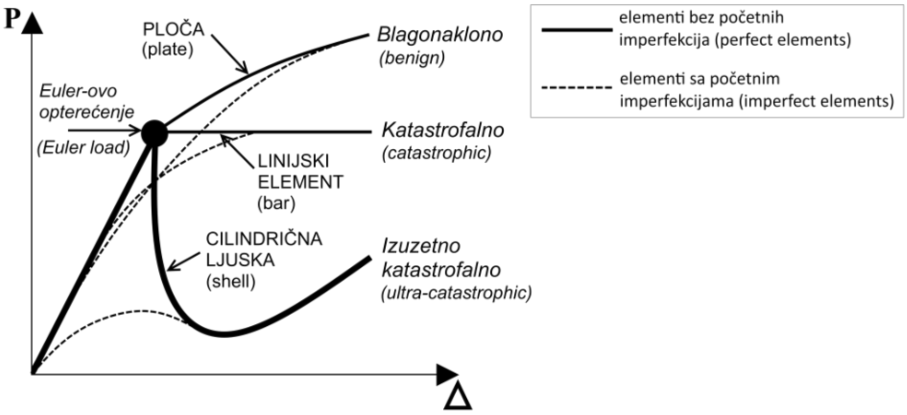
In civil engineering, buckling of cylindrical shell is a relevant criterion for design of chimneys, wind turbine towers, silos and similar structures. It is necessary to find a balance between safety and cost-effectiveness, but to remain in the financially justified limits at the same time, using the less conservative design procedures that will retain safety in terms of design resistance and stability. This is the aim of the paper, i.e. the influence of different values of amplitudes of initial geometrical imperfections, set in the form of the first eigenmode, on shell behaviour with the analysis of the choice of materials from which the shells are made.

The results of the numerical analysis of the stability of cylindrical and conical shells using software package Abaqus are presented in the paper. The influence of material and geometric nonlinearity on critical buckling stress and buckling resistance is analysed. Material nonlinearity has been taken into calculation using the experimental stress-strain curves of carbon and stainless steel, while the geometric nonlinearity has been included through various levels of initial imperfections. The results of the conducted numerical analysis have been compared with the recommendations given in EN 1993-1-6 [9] and the values of the reduction factor, calculated as ratio between critical buckling stress and buckling resistance have been defined. The values of the reduction factor are compared with the data available in the reference literature.

2 AXIALLY COMPRESSED CYLINDRICAL AND CONICAL SHELLS

During the recent decades, a large number of researches have been focused on the analysis of shell stability problem. The case of axially loaded cylindrical shells exposed to compression stands out as characteristic for this type of structural elements, and it is analysed in this paper. The reason lies in considerably different behaviour of cylindrical shells under axial load than of the plates and columns [3]. This is graphically presented in figure 1 for different structural elements exposed to axial compression. The force-strain curves obtained by the analysis for the initial phase, ultimate phase – where the loss of stability occurs and post-buckling phase are given. In figure 1, for perfect plates the load *(P)* can actually increase above the Euler buckling load with increasing axial deformation (Δ). For perfect bars the load (*P*) neither increases nor decreases, while for perfect shells with further increase of deformation (Δ) the load (*P*) decreases beyond the Euler buckling load. When considering the influence of initial imperfections, the Euler load for bars is the maximum load for all axially compressed elements, with and without initial imperfections. The plate shows an increase in deformation (Δ) in the case of an element koje je veće od Eulerovog opterećenja. Cilindrična ljuska s početnim imperfekcijama gubi stabilnost pri opterećenju znatno manjem od Eulerovog opterećenja, čime je ponašanje ovog elementa okarakterisano kao izuzetno katastrofalno [10].

with initial imperfections, but in both cases they can accept the load that is greater than Euler's. A cylindrical shell with initial imperfections loses stability at a load significantly smaller than Euler's which has been characterized as extremely catastrophic behaviour [10].



Slika 1. Krive sila–deformacija za različite pritisnute konstruktivne elemente [10]
Figure 1. The force-strain curves for different axially compressed structural elements [10]

2.1 Linearno-elastična teorija stabilnosti – kritično opterećenje

Ponašanje kružnih cilindričnih ljuski moguće je opisati jedinstvenim setom jednačina, ali zbog svoje kompleksnosti, bez praktične su primene. Iz navedenih razloga, uvode se određena uprošćenja, zanemarujući veličine koje imaju mali uticaj na razmatrani fenomen. Jedan takav set jednačina za opisivanje ponašanja ljuski predložio je Donnell [6]. U slučaju aksijalno opterećenih ljuski, set jednačina moguće je svesti na jednu jednačinu osmog reda, koja je poznata kao Donnell-ova jednačina i može se koristiti za određivanje kritičnog opterećenja – kako usled aksijalnog pritiska, tako i usled torzije i unutrašnjeg pritiska. Kritičnu vrednost opterećenja moguće je dobiti rešavanjem Donnell-ove jednačine i jedno od rešenja dao je Batdorf [2] za slobodno oslonjen cilindar na oba kraja. Dobijena vrednost kritičnog napona, σ_{cr}, u literaturi još nazivana i klasično rešenje, prikazana je jednačinom (1):

2.1 Linear-elastic theory – critical load

The behaviour of cylindrical shells can be described by a single set of equations, but due to its complexity they are impracticable. Therefore, certain simplifications are made by ignoring the parameters that have a small influence on the considered phenomenon. One such set of equations for description of shell behaviour was proposed by Donnell [6]. In the case of axially loaded shells, the set of equations can be reduced to one equation of the eighth order, which is known as Donnell's equation and which can be used for determining the critical load, both under the axial compression and to the torsion and internal pressure. The critical load value can be obtained by solving the Donnell's equation and such a solution was provided by Batdorf [2] for a cylinder pinned on both ends (pinned). The obtained solution, alternatively called the classical solution in the literature, is presented by the eq. (1):

$$\sigma_{cr} = \frac{1}{\sqrt{3(1-\mu^2)}}\frac{Et}{R} \tag{1}$$

gde je:
- E – modul elastičnosti;
- t – debljina ljuske;
- μ – Poisson-ov koeficijent;
- R – poluprečnik krivine cilindra.

Jednačina (1) primenjiva je na cilindre srednje dužine, koji su najzastupljeniji u praksi, pa shodno tome i veoma važni.

U slučaju konusnih ljuski, teorijske analize [13] pokazale su da se kritično opterećenje, P_{cr}, može izraziti na sledeći način:

where:
- E – elastic modulus;
- t – shell thickness;
- μ – Poisson's ratio in elastic range;
- R – radius of curvature.

The eq. (1) is applicable for cylinders of medium length which are the most common in practice, and therefore very important.

In the case of the conical shells, theoretical analyses [13] demonstrated that the critical load can be expressed in the following way:

$$P_{cr} = \frac{2\pi E t^2 \cos^2\alpha}{\sqrt{3(1-\mu^2)}} \tag{2}$$

gde je:

E – Young-ov modul;
t – debljina ljuske;
μ – Poisson-ov koeficijent;
α – nagib izvodnice konusa u odnosu na vertikalnu osu.

Jednačina (2) primenjiva je za vrednost ugla α između 10° i 75°. Za uglove manje od 10° može se uzeti vrednost koja odgovara cilindričnim ljuskama iste visine i debljine, a poluprečnika osrednjene vrednosti poluprečnika konusa.

2.2 Lom i postkritično ponašanje

Eksperimentalna ispitivanja ljuski pokazala su da je kritičan napon – pri kome dolazi do loma – često znatno manji od teorijski dobijenog primenom linearno elastične teorije. Stoga, može se izvesti zaključak da je ona neadekvatna za opisivanje ponašanja aksijalno pritisnutih cilindara i da se opisivanje relanog ponašanja može postići uvođenjem materijalne nelinearnosti u teoriju velikih deformacija.

Značajan napredak u razumevanju ponašanja cilindričnih ljuski napravio je Donnell 1934. godine [7]. On je uvideo važnost primene nelinearne teorije, odnosno značaj iznalaženja ne samo opterećenja pod kojim dolazi do bifurkacione stabilnosti cilindrične ljuske, već i postkritičnog ponašanja ljuski. Eulerova kritična sila na slici 1 predstavlja tačku bifurkacione stabilnosti, nakon čijeg dostizanja se funkcija ravnoteže menja. Zbog velikih uprošćenja, dobijeni rezultati nisu bili primenljivi. Dalja istraživanja bila su usmerena ka detaljnijem opisivanju ponašanja cilindra u postkritičnoj oblasti, kao i ispitivanjima uloge početnih imperfekcija u ponašanju cilindra. Izdvojiće se rad Donnell-a i Wan-a iz 1950. godine [8]. Oni su definisali diferencijalne jednačine kojima se opisuje ponašanje cilindričnih ljuski s početnim imperfekcijama. Grafički prikaz rešenja dat je na slici 2a. Sa A_0/t na slici definisan je odnos amplitude početne imperfekcije prema debljini ljuske t. Iako su dalja eksperimentalna ispitivanja pokazala da su dobijeni rezultati manje tačni, izvedeni su sledeći zaključci:

where:

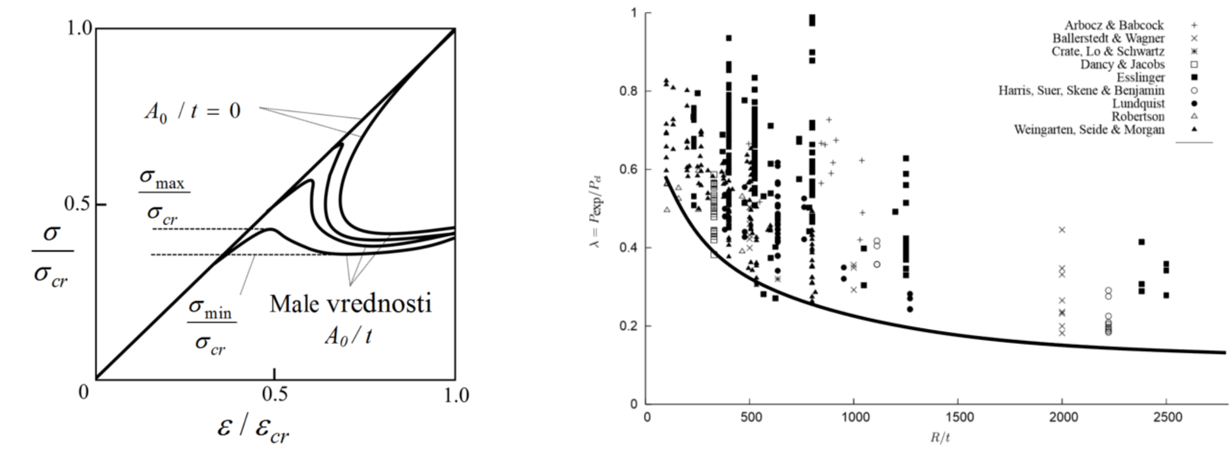
E – elastic modulus;
t – shell thickness;
μ – Poisson's ratio in elastic range;
α – semi-vertex angle of cone.

The eq. (2) is applicable for the value of angle α between 10° and 75°. For the angles smaller than 10° one can assume a value corresponding to the cylindrical shells of the same height and thickness, whose radius is an averaged value of the cone radius.

2.2 Failure and post-buckling behaviour

Experimental research showed that the critical buckling stress at which failure occurs is often considerably lower than the theoretically calculated one obtained using the linear-elastic theory. Therefore, it can be concluded that the linear-elastic theory is inadequate for description of behaviour of axially compressed cylinders, and that describing actual behaviour can be accomplished using the nonlinear large deformation theory.

A considerable advance in understanding cylindrical shell behaviour was made by Donnell in 1934 [7]. He grasped the importance of implementation of nonlinear theory, i.e., the importance of not only finding the load under which the bifurcation stability of a cylindrical shell occurs, but also learning about the post-buckling behaviour of the shells. Euler's critical force in figure 1 represents the point of bifurcation stability after which the equilibrium function changes. Due to the large simplifications, the obtained results were not applicable. Further research was directed towards a more detailed description of the behaviour of the cylinder in the post-critical field, as well as the role of initial imperfections in its behaviour. The work of Donnell and Wan of 1950 will be highlighted [8]. They defined differential equations describing behaviour of cylindrical shells with initial imperfections. The graphical presentation of the solution is provided in figure 2a. With A_0/t, the relationship between the amplitude of the initial imperfections and the shell thickness t is defined in the figure. Although the



a) b)

Slika 2. a) Efekti imperfekcija na postkritično ponašanje cilindara [8]; b) Rezultati ispitivanja aksijalno pritisnutih izotropnih cilindričnih ljuski [4]

Figure 2. a) Effect of imperfections on post-buckling behaviour of cylinders [8], b) Test data for axially compressed isotropic shells [4]

– i najmanja imperfekcija vodi do znatnog smanjenja vrednosti kritičnog napona;

– minimalna vrednost napona (σ_{min}) nije pod velikim uticajem veličine imperfekcije, pa bi usvajanje ove vrednosti, kao kritične, dovelo do konzervativnog rešenja za projektante.

Do rešenja koje je veoma doprinelo rešavanju problema u praksi došao je Koiter [11], koristeći uprošćenu teoriju velikih deformacija na primeru aksisimetričnih početnih imperfekcija. Na slici 2b prikazani su rezultati eksperimentalnih ispitivanja cilindričnih ljuski, koje su objavili različiti autori. Na apscisi je označen odnos prečnika (R) i debljine ljuske (t), dok je na ordinati definisana normalizovana vrednost kritičnog opterećenja (λ), koja predstavlja odnos kritičnog opterećenja dobijenog eksperimentalnim putem (P_{exp}) i opterećenja dobijenog primenom klasičnog rešenja (P_{cl}). Punom linijom na grafiku obeleženo je rešenje koje je dobijeno množenjem opterećenja dobijenog klasičnom teorijom s korekcionim faktorom definisanim u proračunskim preporukama NASA-e [14]. Utemeljeno je objašnjenje da glavni razlog za veliku disperziju rezultata eksperimentalnih ispitivanja u odnosu na teoriju predstavljaju početne imperfekcije.

2.3 Proračunske preporuke

Pri projektovanju tankih cilindričnih ljuski, u inženjerskoj praksi koriste se različiti standardi i priručnici. Svi oni u osnovi primenjuju klasično rešenje dobijeno linearno-elastičnom teorijom (jednačina (1)), a zatim ga množe odgovarajućim redukcionim faktorom (*knockdown factor*), kako bi dobili vrednost opterećenja, koju cilindar može da prenese. Ovakva vrednost opterećenja određuje se na osnovu pune zakrivljene linije kojom je obeležena donja granica rezultata ispitivanja na slici 2b.

Postoje različite analitičke formulacije redukcionog faktora za izotropne cilindrične ljuske, aksijalno opterećene. Neke od prvih preporuka potiču s početka 20. veka, kao što su NASA SP-8007 [14]. Prema njima, kritičan napon dobija se iz sledeće formule:

further research proved that the obtained results are less accurate, the following conclusions were drawn:

– even the slightest imperfection causes a considerable lower critical stress value;

– the minimal value is not significantly impacted by the size of the imperfection, so adopting this value as a critical one would lead to a conservative solution for the designers.

The solution which greatly contributed to solving the problems in practice was discovered by Koiter [11] using a simplified large deformation theory on the example of axisymmetric initial imperfections. The obtained result has a good agreement with the lower values obtained by experimental research. In figure 2b the results of the experimental investigation of cylindrical shells published by various authors are presented. The ratio of the diameter (R) and the shell thickness (t) is indicated on the abscissa, while the normalized critical load (λ) is defined on the ordinate, which represents the ratio of the critical load obtained by the experiments (P_{exp}) and the load obtained using the classical solution (P_{cl}). The full line on the graphic is a solution obtained by multiplying the load obtained by the classical theory with the correction (knockdown) factor defined in NASA's design recommendations [14]. An explanation for a large dispersion of the results of experimental tests, in comparison with the theory was established and initial imperfections were allocated as the main reason.

2.3 Design recommendations

Different standards and manuals used in engineering practice give design recommendations for cylindrical and conical shells. All of them basically implement a classical solution obtained using the linear theory, eq. (1), and then multiply it by the corresponding knockdown factor to obtain the load which can be transferred by the cylinder. This factor is determined based on the full curvature marking the lower boundary of test results in figure 2b.

There are different analytical formulations of the knockdown factor for axially compressed isotropic cylindrical shells. Some of the first recommendations date back to the beginning of 20th century, such as NASA SP-8007 [14]. According to them, the critical stress is obtained from the following formula:

$$\sigma_{cr} = \gamma \sigma_{cl} \tag{3}$$

$$\gamma = 1 - 0.901(1 - e^{-\phi}), \qquad \phi = \frac{1}{16}\sqrt{\frac{R}{t}} \tag{4}$$

gde je:

σ_{cl} – kritični napon definisan jednačinom (1);

γ – redukcioni (knockdown) faktor definisan jednačinom (4);

t – debljina ljuske;

R – poluprečnik krivine cilindra.

Da bi se dobile formule (3) i (4), rezultati eksperimentalnih ispitivanja su grubo uzeti u obzir, a da se pritom nije vodilo računa o načinu ispitivanja ili proizvodnje elemenata. Primena je ograničena samo za slobodno oslonjene cilindre na oba kraja. Dodatno, treba biti oprezan s korišćenjem ovih formula za opsege $L/R>5$, jer ne postoje eksperimentalni podaci za ovu

where:

σ_{cl} – critical stress defined by eq. (1);

γ – knockdown factor defined by eq. (4);

R – radius of curvature;

t – shell thickness.

In order to obtain eq. (3) and (4), the results of the experimental tests are roughly taken into account, without considering the method of testing or manufacturing the elements. Application is limited only to shells pinned at both ends. Additionally, one should be cautious about using these formulas for range $L/R>5$ conditions, because there are no experimental data. Taking the influence of imperfection into account, the oblast. Imajući uticaj imperfekcija u vidu, dobijeni kritični napon i dalje je na strani sigurnosti.

U Evropskom standardu [9], koristi se isti izraz za kritičan napon, jednačina (3), ali je redukcioni faktor definisan putem naredne dve jednačine:

obtained critical stress remains on the safety side.

In the European standard [9], the same expression for critical stress, eq. (3), is used, but the reduction factor is defined by the following two equations:

$$\gamma = \frac{0.82}{\sqrt{1+0.01R/t}}, \quad R/t < 212 \tag{5}$$

$$\gamma = \frac{0.70}{\sqrt{0.1+0.01R/t}}, \quad R/t > 212 \tag{6}$$

gde je:

γ – redukcioni (knockdown) faktor;
R – poluprečnik krivine cilindra;
t – debljina ljuske.

Kako bi se isključila mogućnost sveukupnog Eulerovog izvijanja stuba, formule (3), (5) i (6) važe isključivo za cilindre koji ispunjavaju uslov:

where:

γ – knockdown factor;
R – radius of curvature;
t – shell thickness.

In order to exclude the possibility of the overall Euler's buckling of the column, eq. (3), (5) and (6) should be applied exclusively to the cylinders that fulfil the condition:

$$\frac{L}{R} \le 0.95\sqrt{\frac{R}{t}} \tag{7}$$

gde je:

L – dužina cilindra;
R – poluprečnik krivine cilindra.
t – debljina ljuske;

Imperfekcije se u [9] uzimaju u obzir – u skladu sa slikom 3. Dužina poremećaja definisana je izrazom:

where:

L – length of shell;
R – radius of curvature;
t – shell thickness.

Imperfections are taken into account in [9] in accordance with figure 3. The length of the disorder is defined by the expression:

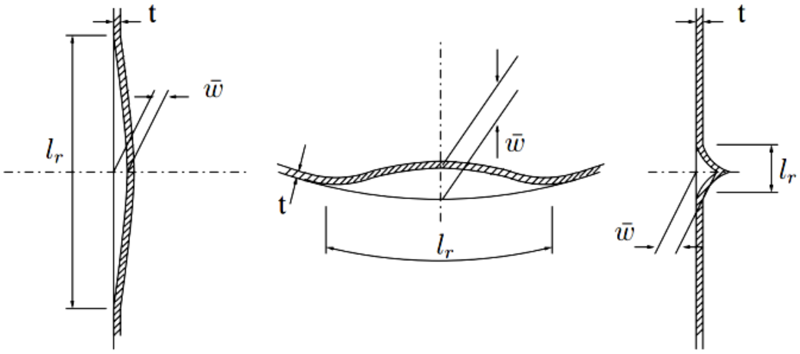
$$l_r = 4\sqrt{Rt} \tag{8}$$

gde je:

R – poluprečnik krivine cilindra.
t – debljina ljuske;

where:

R – radius of curvature;
t – shell thickness.



Slika 3. Merenje dubine $\bar{w}$ početnih imperfekcija [9]
Figure 3. Measurement of depths $\bar{w}$ of initial imperfections [9]

Kada je odnos najveće amplitude $\bar{w}$ u odnosu na odgovarajuće l_r manji od 0,01, treba primeniti formule (5) i (6) za γ; kada je jednak 0,02 vrednosti γ treba uzeti u iznosu od 50%, a između raditi interpolaciju. Kada je ovaj odnos veći od 0,02, ne postoje nikakve preporuke, sugerišući time da bi takve elemente trebalo izbaciti iz primene.

Oba rešenja i preporuke u EN 1993-1-6 [9], kao i preporuke koje je dala NASA-a u SP8007 [14], u osnovi su potekle iz rada Weingarten-a [17]. Osnovna razlika

When the ratio of the largest amplitude $\bar{w}$ relative to the corresponding l_r is less than 0,01, eq. (5) and (6) should be applied to γ; when equal 0,02 values of γ should be taken in the amount of 50%, and between the interpolations should be done. When this ratio is greater than 0,02, there are no recommendations, suggesting that such elements should be excluded from the application.

Both solutions, the recommendations in EN 1993-1-6 [9] and the recommendations provided by NASA in između njih jeste to što je u Evrokodu uzet u obzir kvalitet proizvodnje kroz proizvodne klase (A, B ili C). Druga velika razlika ogleda se u preporukama Evrokoda da se primeni dodatni faktor sigurnosti od 4/3 za aksijalno pritisnute cilindre.

Kod konusnih ljuski, vrednost redukcionog faktora za opseg $10° \leq \alpha < 75°$ iznosi 0,33, dok za opseg $\alpha \geq 75°$ on treba da bude proveren eksperimentalnim putem, kako je preporučeno u [15]. Navedeni redukcioni faktor, kao i u slučaju cilindričnih ljuski, množi se s klasičnim rešenjem datim u jednačini (2).

3 NUMERIČKA ANALIZA

S ciljem kvantifikacije uticajnih parametara koji su značajni za nosivost cilindričnih i konusnih ljuski, sprovedena je numerička parametarska analiza metodom konačnih elemenata. Detaljnom komparativnom analizom, izvršena je procena usklađenosti numeričkih rezultata s rezultatima proračunskih modela koji su dati u evropskom standardu EN 1993-1-6 [9]. Numerička analiza cilindričnih i konusnih tankih ljuski urađena je korišćenjem programa Abaqus, verzija 6.12-3 [1]. U numeričkoj analizi problema stabilnosti, korišćene su dve metode:

– analiza sopstvenih oblika izbočavanja (*LBA – Linear Buckling Analysis*);

– analiza odgovora nakon gubitka stabilnosti ili analiza loma (Postbuckling analysis) primenom GMNIA analize (GMNIA – Geometrically and materially nonlinear analysis with imperfections included).

U inicijalnoj fazi proračuna, koja daje predviđanje sopstvenih oblika izbočavanja, korišćena je linearno-elastična analiza. Analiza stabilnosti elementa primenom GMNIA analize zasniva se na rešavanju nelinearne jednačine ravnoteže, primenom odgovarajuće numeričke metode. U radu je primenjena metoda konstantnog sfernog luka, poznata i kao Riksova metoda [12].

Numerička analiza prikazana u ovom radu obuhvatila je analizu cilindrične ljuske dužine 10 m i poluprečnika 2,5 m, debljine zida ljuske od 6,0 mm do 30,0 mm, koja pripada opsegu cilindričnih ljuski srednje dužine, kako je definisano u EN 1993-1-6, Aneks D [9]. Takođe, numerička analiza obuhvatila je i konusnu ljusku iste dužine i promenjivog poluprečnika od 1,25 m do 2,5 m, debljine zida ljuske 10,0 mm. Modeli cilindričnih i konusnih obostrano zglobno oslonjenih ljuski formirani su u Abaqus-u [1], pomoću površinskih S4R konačnih elemenata. Mreža za sve analizirane modele formirana je od konačnih elemenata S4R aproksimativne veličine 200 x 200 mm, za koju je utvrđeno da rezultati počinju značajnije da konvergiraju (razlika u kritičnom naponu izbočavanja koji odgovara prvom obliku izbočavanja manja je od 0,20%).

Mehanička svojstva materijala definisana su nelinearnom vezom napona i dilatacija dobijenih ispitivanjem pri zatezanju epruveta izrađenih od vruće valjanog profila od ugljeničnog čelika kvaliteta S275 [16] i epruveta od hladno valjanog nerđajućeg čelika austenitne mikrostrukture sa oznakom 1.4301 [5] (kako je prikazano na slici 4a). Zglobno oslanjanje ostvareno je defini-

SP8007 [14] basically originate from the work of Weingarten [17]. The basic difference between them is that Eurocode takes into account the quality of production, through fabrication tolerance quality classes (A, B or C). The other large difference between the European standard [9] and SP8007 [14] is reflected in the Eurocode recommendations to implement an additional safety factor of 4/3 for axially compressed cylinders.

In the case of conical shells, the value of the knock-down factor for the range $10° \leq \alpha < 75°$ amounts to 0,33, while for the range $\alpha \geq 75°$ it must be verified experimentally, as recommended in [15]. The mentioned knock-down factor, as in the case of cylindrical shells, is multiplied with the classical solution provided in the eq. (2).

3 NUMERICAL ANALYSIS

In order to quantify the influence of main parameters on the resistance of cylindrical and conical shells, a numerical parametric analysis was performed using the finite element method. A detail comparative analysis was conducted in order to quantify the compliance of numerical analysis results with design recommendations given in European standard EN 1993-1-6 [9]. Numerical analysis of cylindrical and conical thin-walled shells was performed using the Abaqus software, version 6.12-3 [1]. In the numerical analysis of the stability problem, two methods were used:

– analysis of the linear bifurcation eigenvalue (*LBA - Linear Buckling Analysis*);

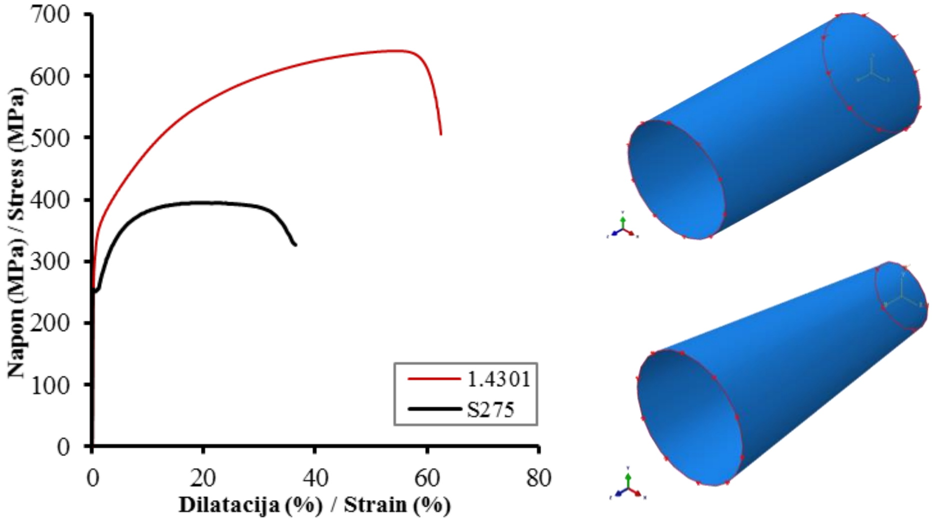
– analysis of response after stability loss or nonlinear analysis (Postbuckling analysis) with GMNIA analysis (GMNIA - Geometrically and materially nonlinear analysis with imperfections included).

A linear elastic analysis was used in the initial phase of the calculation which provides prediction of eigenmodes. The element stability analysis with GMNIA is based on solving the nonlinear equilibrium equation using an appropriate numerical method. An arc length method, also known as the Riks method [12] is implemented in the paper.

The numerical analysis presented in this paper included the cylindrical shell 10 m long with 2,5 m radius, with the wall thickness ranging between 6,0 mm and 30,0 mm, which belongs to the medium long cylindrical shells as defined in EN 1993-1-6, Annex D [9]. In addition, the numerical analysis included the conical shell of variable radius ranging between 1,25 m and 2,5 m, with 10,0 mm wall thickness. The models of cylindrical and conical shells pinned on both ends are defined in Abaqus [1] using shell S4R finite elements. The mesh for all analysed models has been formed using S4R finite elements of the approximate size of 200 x 200 mm for which the results start to converge more considerably (the critical buckling stress which corresponds to the first eigenmode is lower than 0,20%).

Mechanical properties of the material are defined by a nonlinear stress-strain obtained by tensile tests of coupons made of hot-rolled section of carbon steel grade S275 [15] and coupons made of cold-formed stainless steel of austenitic microstructure designated as 1.4301 [5], as given in figure 4a. Pinned supports are realized by defining the boundary conditions along the sanjem graničnih uslova po obimu ljuske, i to na jednom kraju sprečavanjem pomeranja u pravcu sve tri glavne ose U1=U2=U3=0, a na suprotnom kraju elementa na kojem se nanosi opterećenje dozvoljeno je pomeranje u pravcu globalne Z-ose, što odgovara podužnoj X-osi ljuske (U1=U2=0), kako je prikazano na slici 4b. Opterećenje u obliku aksijalnog pritiska nanosi se na jednom kraju ljuske, kao raspodeljeno opterećenje svuda po obimu.

shell circumferences. On one end of the shell restraining displacement in the direction of all three main axes U1=U2=U3=0 is adopted. On the opposite end on which the load is applied, the displacement in the direction of the global Z-axis is free, which corresponds to the longitudinal X-axis of the shell (U1=U2=0), as displayed in figure 4b. The axial compression loading is applied to one end of the shell as a load distributed along the entire circumference.



a) b)

Slika 4. a) Dijagram napon-dilatacija za ugljenični čelik S275 i nerđajući čelik 1.4301; b) model konusne i cilindrične ljuske i uslovi oslanjanja

Figure 4. a) stress-strain curves for carbon steel S275 and stainless steel 1.4301 b) model of conical and cylindrical shell and boundary conditions

4 REZULTATI NUMERIČKE ANALIZE I POREĐENJE S PREPORUKAMA DEFINISANIM U EN 1993-1-6

Vrednosti elastičnog kritičnog napona izbočavanja za cilindrične ljuske debljine od 6,0 do 30,0 mm i konusne ljuske debljine 10,0 mm definisane su primenom LBA metode u Abaqusu [1] i izrazu definisanom u EN 1993-1-6 [9], što je ujedno i klasično rešenje prikazano jednačinom (1). Materijalna nelinearnost uvedena je u numeričke primere putem stvarne veze napona i dilatacije za dva analizirana materijala, a početna geometrijska imperfekcija zadata je kao pomeranje određene amplitude koje odgovara prvom sopstvenom obliku izbočavanja (LBA analiza u Abaqus-u [1]).

Uticaj različitih vrednosti imperfekcija na nosivost na izbočavanje analiziran je na primeru cilindrične i konusne ljuske debljine 10,0 mm, primenom metode materijalne i geometrijske nelinearne analize sa imperfekcijama (*GMNIA*).

4 NUMERICAL ANALYSIS RESULTS AND COMPARISON WITH RECOMMENDATIONS GIVEN IN EN1993-1-6

The values of the elastic critical buckling stress for cylindrical shells with thickness from 6,0 mm to 30,0 mm and conical shell with 10,0 mm thickness are defined using the LBA analysis in Abaqus [1] and according to design recommendations given in EN 1993-1-6 [9] which is at the same time the classical solution presented in the eq. (1). Material nonlinearity is introduced in the numerical models through the real stress-strain relation for two analysed materials, and the geometrical imperfection is set as displacement of a certain amplitude, which corresponds to the first eigenmode (LBA analysis in Abaqus [1]).

The impact of different values of imperfections on the critical buckling stress is analysed on the example of cylindrical and conical shells 10,0 mm thick, with the geometrically and materially nonlinear analysis with imperfections included (*GMNIA*).

4.1 Rezultati numeričke analize – cilindrične ljuske debljine 10 mm

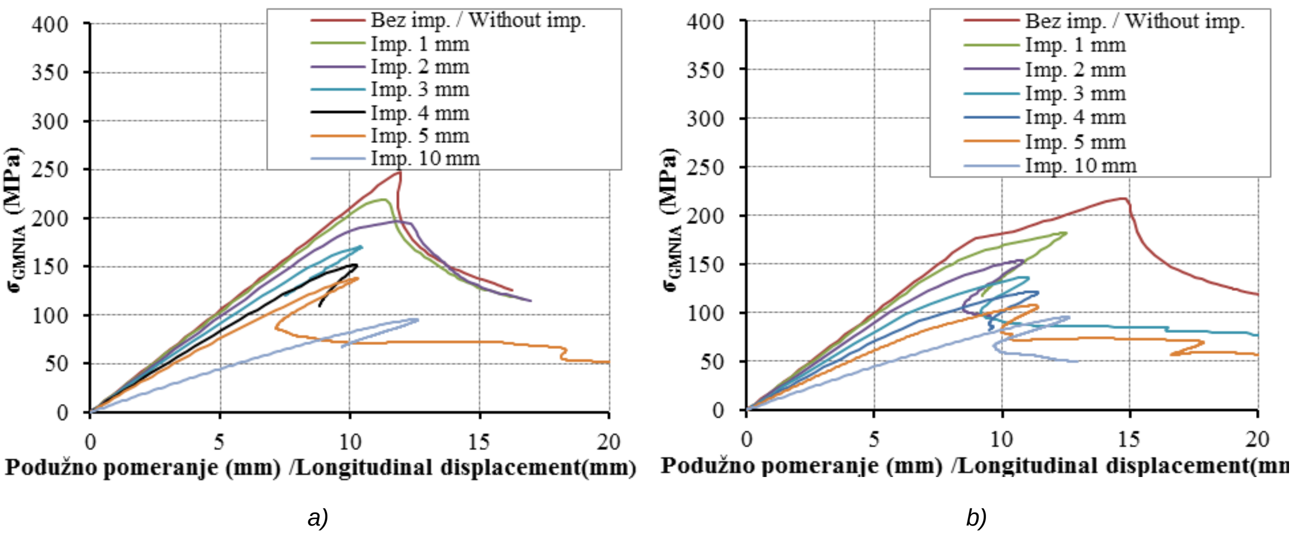
Na slici 5 prikazana je nosivost na izbočavanje cilindrične ljuske od ugljeničnog čelika S275 i nerđajućeg čelika 1.4301.

Cilindrična ljuska od nerđajućeg čelika ima 12,5% manju nosivost na izbočavanje u poređenju sa istim cilindrom od ugljeničnog čelika, za model bez početnih imperfekcija (MNA – *materially nonlinear analysis*). Kritičan napon izbočavanja cilindrične ljuske debljine 10,0 mm od ugljeničnog i nerđajućeg čelika, dobijen primenom LBA metode u Abaqusu [1], iznosi 509,3 MPa i 485,0 MPa, respektivno.

4.1 Results of numerical analysis - cylindrical shells with thickness of 10 mm

Buckling resistance of the cylindrical shell of S275 carbon steel and 1.4301 stainless steel is given in figure 5.

The cylindrical shell of stainless steel has 12,5% lower buckling resistance in comparison with the same cylinder made of carbon steel, regarding the numerical model without initial imperfections (*MNA - materially nonlinear analysis*). Critical buckling stress obtained from LBA analysis in Abaqus [1], for cylindrical shell made from carbon and stainless steel with 10,0 mm thickness of 509,3 MPa and 485,0 MPa, respectively.



Slika 5. Nosivost na izbočavanje cilindrične ljuske t=10,0 mm za različite vrednosti početne imperfekcije: a) ugljenični čelik S275; b) nerđajući čelik 1.4301

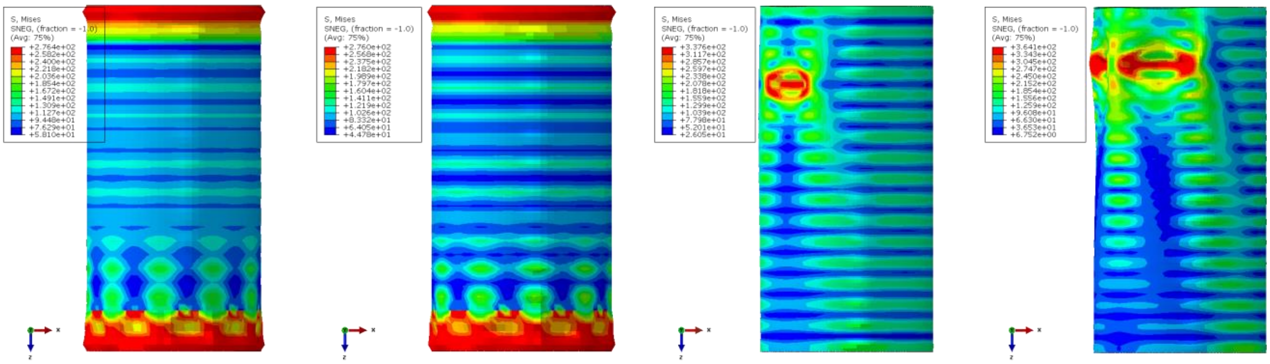
Figure 5. Buckling resistance of cylindrical shell t=10,0 mm for different initial imperfektions: a) carbon steel S275, b) stainless steel 1.4301

Kod cilindričnih ljuski od ugljeničnog čelika, imperfekcije od 1,0 i 2,0 mm izazivaju aksijalno simetrično izbočavanje ljuske u zoni neposredno uz oslonce, odnosno neposredno ispod zone unošenja opterećenja, kao na slici 6a i 6b. Oblik izbočavanja cilindra od nerđajućeg čelika, pri manjim vrednostima imperfekcija, u obliku je nesimetričnog izbočavanja (*dimple buckling*) u zoni unošenja opterećenja, koje je prikazano na slici 6c i 6d, a koje se – kao karakterističan oblik izbočavanja – zadržava i pri povećanju imperfekcija do 10,0 mm.

Takođe, kod nerđajućeg čelika, u oblasti napona između napona proporcionalnosti f_p i konvencionalne granice razvlačenja f_{02} javlja se progresivni pad tangentnog modula elastičnosti E_t, što utiče i na znatno smanjenje krutosti koja je uočljiva na slici 5b, kod cilindričnih ljuski od nerđajućeg čelika.

In the case of the cylindrical shells of carbon steel, the imperfections of 1,0 and 2,0 mm cause axially symmetrical buckling in the zone immediately next to supports, i.e. immediately below the zone where the load is applied, as given in figures 6a and 6b. The response of the stainless steel cylindrical shell at lower values of imperfections has the form of dimple buckling in the zone where the load is applied, as displayed in figures 6c and 6d, which is a characteristic response of the structure retained even when the imperfections are increased up to 10,0 mm.

In addition, in the case of stainless steel, in the stress range between the proportionality limit stress f_p and the 0.2% proof stress f_{02} there is a progressive decline of tangent elastic modulus E_t, which causes the considerable decrease of stiffness as observable in figure 5b, in cylindrical shells of stainless steel.



a) b) c) d)

Slika 6. Izbočavanje cilindrične ljuske debljine od 10,0 mm za različite vrednosti imperfekcija: a) 1 mm - S275; b) 2 mm - S275; c) 1 mm - 1.4301; d) 2 mm - 1.4301

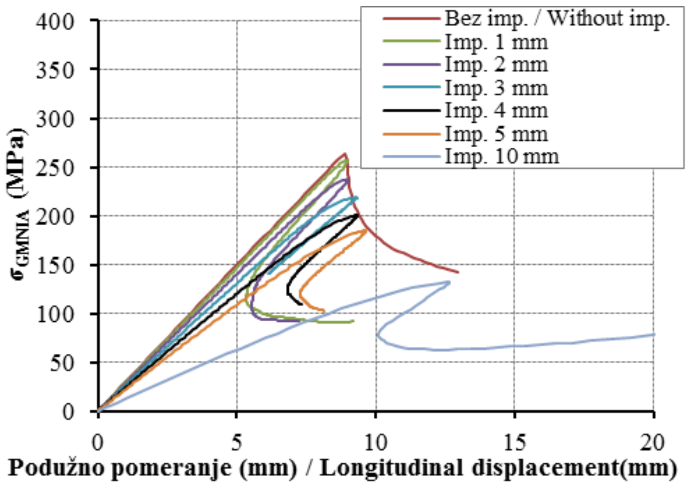
Figure 6.Buckling of cylindrical shell 10,0 mm thick for different values of imperfections - a) 1 mm – S275, b) 2 mm – S275, c) 1 mm – 1.4301, d) 2 mm - 1.4301

4.2 Rezultati numeričke analize – konusne ljuske debljine 10 mm

Uticaj različitih vrednosti početnih geometrijskih imperfekcija zadatih u obliku prvog sopstvenog oblika izbočavanja na nosivost na izbočavanje konusne ljuske od ugljeničnog čelika S275 prikazan je na slici 7.

4.2 Results of numerical analysis - conical shells with thickness of 10mm

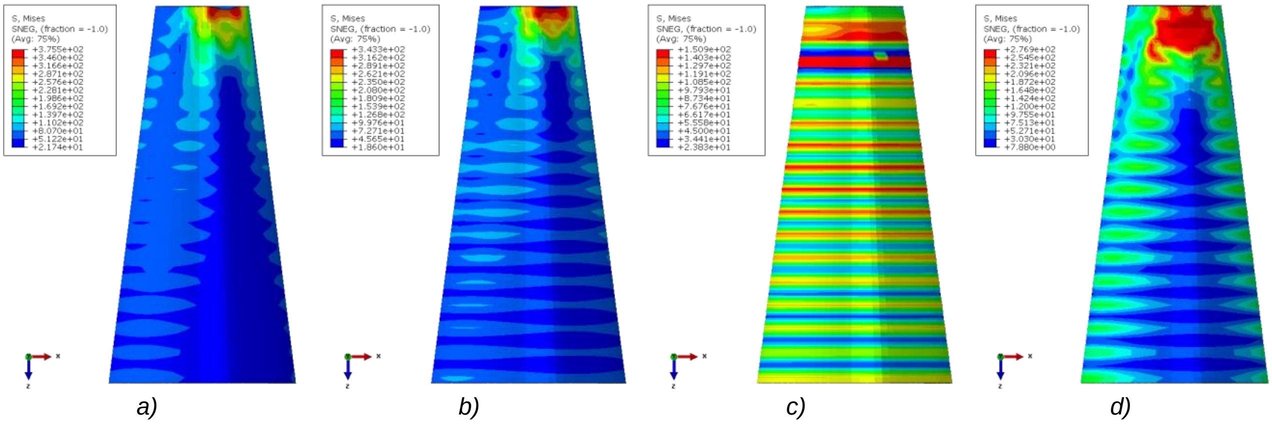
The impact of different values of initial geometrical imperfections on the buckling resistance of the conical shell made of S275 carbon steel is given in figure 7.



Slika 7. Nosivost na izbočavanje konusne ljuske t=10,0 mm za različite vrednosti početne imperfekcije
Figure 7. Buckling resistance of conical shell t=10,0 mm for different initial imperfektions

Kritičan napon izbočavanja konusne ljuske od ugljeničnog čelika debljine 10,0 mm, dobijen primenom LBA metode u Abaqusu [1], iznosi 504,3 MPa. Odgovor analizirane konusne ljuske jeste u obliku nesimetričnog izbočavanja (*dimple buckling*) u zoni unošenja opterećenja, koje se kao karakterističan odgovor konstrukcije zadržava i pri povećanju imperfekcija do 10 mm (kako je prikazano na slici 8).

Critical buckling stress obtained from LBA analysis in Abaqus [1] is 504,3 MPa for conical shell made of carbon steel with 10,0 mm thickness. The response of a conical shell made of carbon steel is in a form of dimple buckling in the zone where the load is applied which is a characteristic response of the structure retained even when the imperfections are increased up to 10,0 mm, as displayed in figure 8.



Slika 8. Izbočavanje konusne ljuske debljine od 10,0 mm od ugljeničnog čelika S275 za različite vrednosti imperfekcija: a) 1 mm; b) 2 mm; c) 3 mm; d) 4 mm

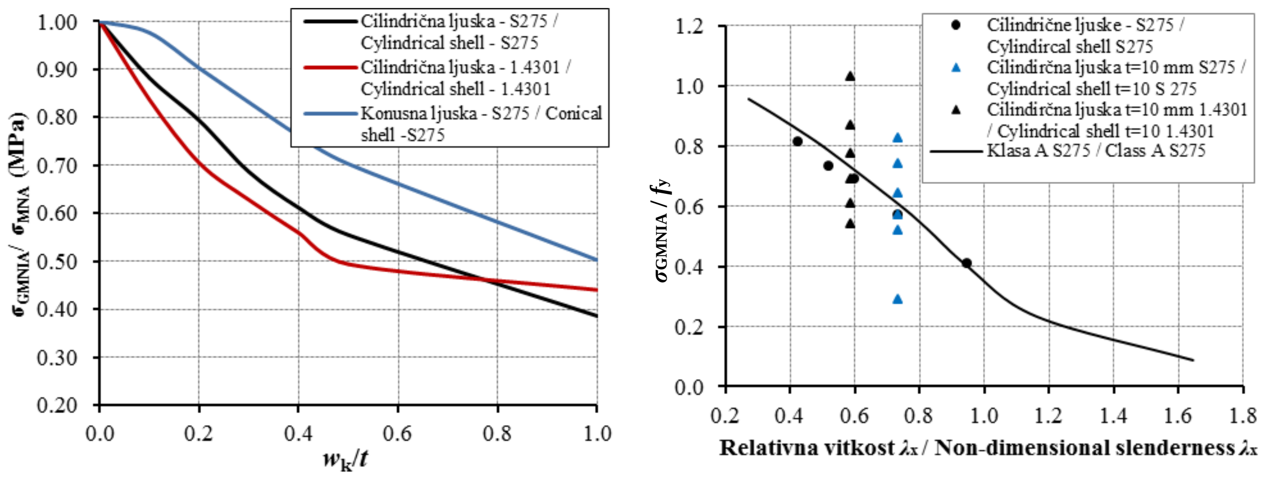
Figure 8. Buckling of cylindrical shell 10,0 mm thick made of S275 carbon steel for different values of imperfections: a) 1 mm, b) 2 mm, c) 3 mm, d) 4 mm

4.3 Poređenje dobijenih rezultata s proračunskim preporukama i literaturom

Redukcija nosivosti cilindrične i konusne ljuske od dva analizirana materijala debljine 10,0 mm, usled povećanja geometrijskih imperfekcija, prikazana je na slici 9a. Na horizontalnoj osi prikazan je odnos zadate početne geometrijske imperfekcije i debljine ljuske (w_k/t). Na vertikalnoj osi prikazan je odnos nosivosti ljuske na izbočavanje usled materijalne nelinearnosti i zadatih početnih geometrijskih imperfekcija (σ_{GMNIA}) i bez početnih geometrijskih imperfekcija (σ_{MNA}). Imperfekcija od 1,0 mm izaziva redukciju nosivosti od 12% odnosno 17%, za ugljenični čelik i nerđajući čelik kod cilindričnih ljuski, respektivno. Međutim, ova redukcija nosivosti kod konusnih ljuski od ugljeničnog čelika za istu vrednost imperfekcije znatno je manja i iznosi 2%. Takođe, povećanjem početnih geometrijskih imperfekcija do 5,0 mm, redukcija nosivosti dostiže i do 45% kod ugljeničnih čelika, odnosno 50% kod nerđajućeg čelika u slučaju cilindričnih ljuski i 30% u slučaju konusnih ljuski od ugljeničnih čelika. Cilindrične i konusne ljuske od ugljeničnog čelika pokazuju ujednačen pad nosivosti pri povećanju imperfekcija, ali sa značajnom kvantitativnom razlikom u pogledu vrednosti redukcije nosivosti. S druge strane, cilindrična ljuska od nerđajućeg čelika pri povećanju imperfekcija od 5,0 mm do 10,0 mm pokazuje manji pad nosivosti, od 51% do 57% (kako je prikazano na slici 9a), dok je kod konusne ljuske od ugljeničnog čelika povećanje geometrijskih imperfekcija za istu vrednost rezultovalo padom nosivosti od 20%. Konusne cilindrične ljuske od ugljeničnog čelika pokazuju manji pad vrednosti napona izbočavanja u poređenju s cilindričnim ljuskama, u proseku 10% za istu vrednost početnih geometrijskih imperfekcija.

4.3 Comparison of the obtained results with design recommendations and literature

The reduction of buckling resistance of cylindrical and conical shell made of two analysed materials with 10,0 mm thickness caused by the increase of geometrical imperfections is presented in figure 9a. The horizontal axis represent the initial geometrical imperfections amplitude vs. shell thickness ratio (w_k/t). The vertical axis represent the ratio between buckling resistance shell due to geometrically and materially nonlinear analysis with imperfections included (σ_{GMNIA}) and buckling resistance without imperfections (σ_{MNA}). An imperfection of 1,0 mm causes the buckling resistance reduction of 12%, i.e. 17%, for carbon steel and stainless steel in cylindrical shells, respectively. However, this reduction of buckling resistance of conical shells of carbon steel for the same value of imperfection is considerably lower and amounts to 2%. In addition, by increasing the initial geometrical imperfections up to 5,0 mm, the reduction of buckling resistance is 45% for carbon steel, i.e. about 50% for stainless steel in the case of cylindrical shells and 30% in the case of conical shells of carbon steel. Cylindrical and conical shells made of carbon steel demonstrate a uniform decline of buckling resistance as the imperfections increase, but with a considerable quantitative difference in terms of the values of buckling resistance reduction. On the other hand, a cylindrical shell made of stainless steel, when the imperfections are increased from 5,0 mm to 10,0 mm exhibits a small decline of buckling resistance, from 51% to 57%, as presented in figure 9a. In the case of a conical shell made of carbon steel, the increase of geometrical imperfections for the same value resulted in the buckling resistance drop of 20%. Conical cylindrical shells made of carbon steel show a lower drop of buckling stress value in respect to the cylindrical shells, which is 10% in average for the same value of initial geometrical imperfections.



a) redukcija napona izbočavanja za ljuske debljine 10 mm
a) reduction of bucking stress of 10 mm thick shells

b) rezultati numeričke analize
b) comparison of the results of the numerical analysis with design recommendations

Slika 9. Poređenje rezultata numeričke analize s preporukama za proračun
Figure 9. Impact of geometrical imperfections

Poređenje rezultata numeričke analize za cilindrične ljuske od ugljeničnog i nerđajućeg čelika s proračunskim preporukama definisanim u EN 1993-1-6 [9], za krivu izvijanja definisanu za klasu A proizvodnih tolerancija, prikazano je na slici 9b. Analizirane su clindrične ljuske od čelika S275 debljine od 6,0 mm do 30,0 mm i od nerđajućeg čelika 1.4301 debljine 10,0 mm. Prema EN 1993-1-6 [9], početna geometrijska imperfekcija w_k određuje se u funkciji poluprečnika ljuske i njene debljine, kako je definisano jednačinom (8), kao i parametra lokalne izbočine $U_{0,max}$=0,006 (*dimple tolerance parameter*). Vrednosti zadatih početnih geometrijskih imperfekcija prikazane su u Tabeli (1). Ostvareno je dobro poklapanje rezultata numeričke analize za analizirane debljine cilindrične ljuske i proračunskih preporuka za klasu A proizvodnih tolerancija. Takođe, na slici 9b prikazani su rezultati numeričke analize za cilindrične ljuske od ugljeničnog i nerđajućeg čelika debljine 10,0 mm i početnih geometrijskih imperfekcija u opsegu od 1,0 do 10,0 mm.

Za cilindričnu ljusku debljine 10,0 mm od ugljeničnog čelika S275 i geometrijske imperfekcije do 4,0 mm rezultati numeričke analize zadovoljavaju empirijski definisane preporuke u EN 1993-1-6 [9], kako je prikazano na slici 9b. Za imperfekcije veće od 4,0 mm, što su ujedno i vrednosti imperfekcije koju preporučuje Evrokod za analiziranu cilindričnu ljusku i klasu A proizvodnih tolerancija (Tabela [1]), rezultati numeričke analize ne zadovoljavaju proračunske preporuke. Iako proračunske preporuke za ljuske od nerđajućeg čelika još uvek nisu definisane, analiza ponašanja ljuski od ovoga materijala bila je posebno značajna zbog specifičnosti njegovih mehaničkih svojstava. Kada se mehanička svojstva nerđajućeg čelika primene u proračunskim preporukama za ljuske, datim u EN 1993-1-6 [9], uočava se takođe da se najbolja poklapanja rezultata numeričke analize, sa ovako definisanim proračunskim preporukama, dobija za imperfekcije do 4,0 mm (slika 9b). Svakako, definisanje jasnih proračunskih preporuka u ovoj oblasti zahteva opsežna numerička i eksperimentalna ispitivanja.

Comparison of the results of the numerical analysis for cylindrical shells from carbon and stainless steel and the design recommendations given in EN 1993-1-6 [9] for the buckling curve defined for class A of fabrication tolerances is given in figure 9b. The analysis included cylindrical shells with thicknesses in the range from 6,0 mm to 30,0 mm made from carbon steel S275 and shell made from stainless steel 1.4301 with 10,0 mm thickness. According to EN 1993-1-6 [9] initial geometrical imperfection w_k should be determined as a function of shell radius, thickness and dimple tolerance parameter $U_{0,max}$=0,006, as given in eq. (8). The values of initial geometrical imperfections are presented in Table (1). The suitable prediction of results is achieved for analysed cylindrical shells and design recommendations according to class A of fabrication tolerances. Besides, the results of numerical analysis for cylindrical shells with 10,0 mm thickness made form carbon and stainless steel with initial geometrical imperfections in the range from 1,0 mm to 10,0 mm are presented in figure 9b.

The results of numerical analysis satisfy empirically defined recommendations in EN 1993-1-6 [9] as shown in figure 9b in the case of cylindrical shell with 10,0 mm thickness made of S275 carbon steel with geometrical imperfections up to 4,0 mm. The results of numerical analysis fail to satisfy the design recommendations in the case of imperfections larger than 4,0 mm which is at the same time the value of imperfection recommended by the Eurocode for the analysed cylindrical shell and class A of fabrication tolerances (Table 1). Even though the design recommendations for the stainless steel shells have not been defined yet, the analysis of behaviour of shells made from this material was particularly significant due to the specific mechanical properties of this material. When the mechanical properties of stainless steel are implemented in the design recommendations for shells provided in EN 1993-1-6 [9], one may also observe that the best agreement of the numerical analysis results with such defined design recommendations are obtained for imperfections of up to 4,0 mm (figure 9b). Certainly, defining clear design recommendations in this field requires extensive numerical and experimental research.

Tabela 1. Rezultati parametarske analize – cilindrične ljuske – S275
Table 1. Parametric analysis results – cylindrical shells– S275

t (mm)	Kritičan napon [MPa] / *Elastic critical buckling stress [MPa]*		Nosivost elementa [MPa] / *Buckling resistance [MPa]*				KDF
	EN 1993-1-6 [9] $\sigma_{x,Rcr}=0,605EC_x t/r$	Abaqus [1] σ_{LBA}	EN 1993-1-6 [9] w_k (mm)	EN 1993-1-6 [9] $\sigma_{x,Rk}=\chi_x f_y$	Abaqus [1] σ_{MNA}	Abaqus [1] σ_{GMNIA}	
6	304,9	305,8	3	109,6	230,7	108,2	0,35
8	406,6	407,4	3,4	145,0	243,2	137,7	0,36
10	508,2	509,3	3,8	166,7	247,9	151,6	0,30
12	609,8	610,8	4,2	182,1	250,6	163,8	0,30
15	762,3	762,7	4,6	198,3	253,4	183,4	0,24
20	1016,4	1016,4	5,4	215,6	256,9	193,9	0,19
30	1524,6	1522,2	6,6	234,9	260,0	215,9	0,14

Komparativna analiza rezultata numeričke analize, metodom konačnih elemenata i proračunskih preporuka datih u EN 1993-1-6 [9] za cilindrične ljuske od ugljeničnog čelika S275, debljine od 6,0 do 30,0 mm, prikazana je u Tabeli (1). Geometrijske imperfekcije zadate su prema preporuci za definisanje geometrijske imperfekcije w_k date u [9] za klasu A proizvodnih tolerancija. Iste proračunske preporuke u pogledu zadavanja početnih geometrijskih imperfekcija w_k primenjene su i na konusnu ljusku od ugljeničnog čelika S275, s debljinom zida ljuske od 10,0 mm, kako je prikazano u Tabeli (2).

Takođe, u tabelama (1) i (2), prikazan je odnos nosivosti cilindrične i konusne ljuske, dobijen primenom GMNIA analize u Abaqus-u [1] i kritičnog napona izbočavanja, koji je definisan u EN 1993-1-6 [9], što je definisano kao redukcioni faktor (*KDF - knockdown factor*). Napon izbočavanja, dobijen kao rezultat geometrijske i materijalne nelinearnosti (GMNIA), dobijen u Abaqus-u [1] i primenom proračunskih preporuka u EN 1993-1-6 [9], ima približno iste vrednosti. Razlika u ovako dobijenim rezultatima u opsegu je do 10% za cilindričnu ljusku i 17% za konusnu ljusku od konstrukcionog čelika S275. Takođe, ukoliko se uporede rezultati numeričke MNA analize s rezultatima GMNIA analize u Abaqus-u [1], može se uočiti da se s porastom debljine zida cilindrične ljuske razlika u nosivosti elementa, dobijena na ova dva načina, smanjuje. Može se zaključiti i to da se za kompaktnije preseke, veće debljine zida ljuske, smanjuje uticaj početnih geometrijskih imperfekcija na nosivost elementa, a povećava uticaj materijalne nelinearnosti.

Comparative analysis of results obtained from numerical analysis with design recommendations given in EN 1993-1-6 [9] for cylindrical shells made of S275 carbon steel 6,0 to 30,0 mm thick, is given in Table (1). Comparison of the results is performed in order to define the compliance of numerical analysis with recommendations given in European standard. Geometrical imperfections are defined according to the recommendations for geometrical imperfection w_k provided in EN 1993-1-6, Annex D [9] for class A of fabrication tolerances. The same design recommendations considering definition of geometrical imperfections w_k are also implemented on the conical shell made of S275 carbon steel, with the shell wall thickness of 10,0 mm, as given in Table (2).

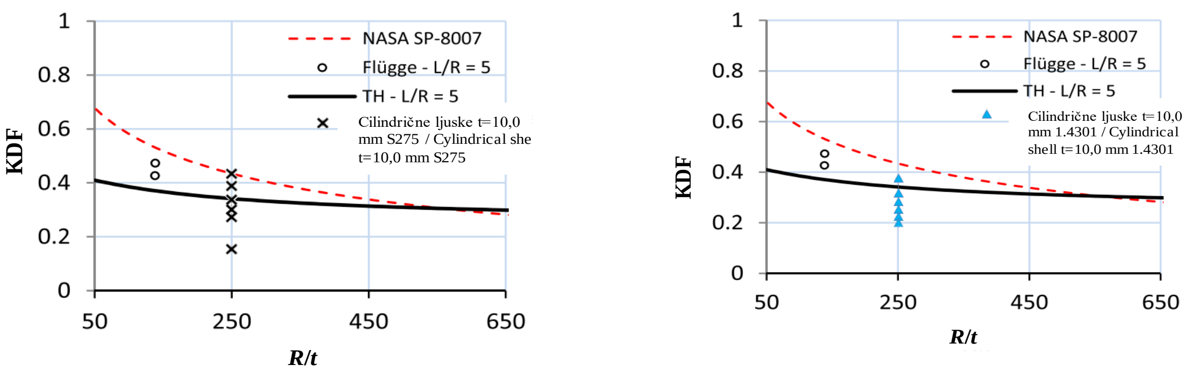
In addition, the relation between buckling resistance of cylindrical and conical shells obtained using the GMNIA analysis in Abaqus [1] and the critical buckling stress which is defined in EN 1993-1-6 [9], defined as knockdown factor - KDF, is given in Table (1) and (2). The buckling resistance obtained as a result of geometrical and material nonlinearity (*GMNIA*) in Abaqus [1] and that obtained using the design recommendations in EN 1993-1-6 [9] have approximately identical values. The difference between obtained results is in the range of 10% for cylindrical shell and 17% for conical shell made form carbon steel S275. When comparing the results of numerical analysis for material nonlinearity (*MNA analysis*) without initial imperfections and the results of an analysis including geometrical and material nonlinearities (*GMNIA*) in Abaqus [1], it can be observed that with the increase of the cylindrical shell thickness, the difference in the buckling resistance obtained by these two methods is reducing. It can be concluded that for the more compact cross-sections, having a higher shell wall thickness, the impact of initial geometrical imperfections on the buckling resistance is decreasing, and the impact of material nonlinearity is increasing.

Tabela 2. Rezultati parametarske analize – konusne ljuske – S275
Table 2. Parametric analysis results – conical shells – S275

t (mm)	Kritičan napon [MPa] / *Elastic critical buckling stress [MPa]*		Nosivost elementa [MPa] / *Buckling resistance [MPa]*				KDF
	EN 1993-1-6 [9] $\sigma_{x,Rcr}=0{,}605EC_x t/r$	Abaqus [1] σ_{LBA}	EN 1993-1-6 [9] w_k (mm)	EN 1993-1-6 [9] $\sigma_{x,Rk}=\chi_x f_y$	Abaqus [1] σ_{MNA}	Abaqus [1] σ_{GMNIA}	
10	504,3	504,8	3,8	166,0	263,3	200,8	0,40

Na slici 10 prikazana su poređenja rezultata numeričke analize za cilindričnu ljusku debljine 10,0 mm, od ugljeničnog i nerđajućeg čelika, čiji rezultati su prikazani na slici 5 (sa geometrijskim imperfekcijama od 1,0 mm do 10,0 mm), s trenutno razvijenim empirijskim preporukama za redukcioni faktor (KDF). Definisane preporuke date su u formi krivih koje pokazuju zavisnost redukcionog faktora i odnosa poluprečnika ljuske i debljine (*R*/*t*). Na slici 10 prikazane su preporuke za redukcioni faktor prema NASA SP-8007 [14], modifikovane krive u zavisnosti od odnosa dužine i poluprečnika ljuske TH - *L*/*R*=5 prema Wagner-u [18], eksperimentalni rezultati prema Flügge-u i rezultati sopstvene numeričke analize. Modifikovana kriva TH - *L*/*R*=5 prema Wagner-u [18] obuhvatila je zajednički uticaj imperfekcija u opterećenju i geometrijskih imperfekcija (*SBPA – single boundary perturbation approach*), i za odnos dužine i poluprečnika ljuske *L*/*R*=5 definiše značajno niže vrednosti redukcionog faktora (KDF) od preporuka datih u NASA SP-8007 [14].

Figure 10 shows comparisons of numerical analysis results for a cylindrical shell of 10,0 mm thick of carbon and stainless steel given in Figure 5 (with geometric imperfections from 1,0 mm to 10,0 mm), with the currently developed empirical recommendations for the knockdown factor (KDF). The defined recommendations are provided in the form of the curves showing dependence of the knockdown factor and ratio of the shell radius and thickness (*R*/*t*). Figure 10 shows recommendations for the reduction factor according to NASA SP-8007 [14], modified curves depending on the ratio of the shell length and radius *L*/*R* according to Wagner [18], experimental results by Flügge and the results of the presented numerical analysis. The modified curve TH - *L*/*R*=5 according to Wagner [18] includes the mutual impact of imperfection in load and geometric imperfection (SBPA - single boundary perturbation approach), and for the ratio of length and radius of shell *L*/*R*=5 defines significantly lower value of the knockdown factor (KDF) from the recommendations given in NASA SP-8007 [14].



a) b)

Slika 10. Redukcioni faktor za cilindrične ljuske: a) ugljenični čelik S275; b) nerđajući čelik 1.4301 [18]
Figure 10. Knockdown factor for cylindrical shells - a) carbon steel S275, b) stainless steel 1.4301 [18]

Cilindrične ljuske od ugljeničnog čelika S275 pokazuju relativno dobro slaganje sa modifikovanom krivom prema Wagner-u [18] za geometrijske imperfekcije definisane u skladu s preporukama datim u EN 1993-1-6 [9] za analiziranu dužinu i prečnik ljuske. Za ljusku debljine 10,0 mm od ugljeničnog čelika S275 i geometrijske imperfekcije od 1,0 do 3,0 mm rezultati numeričke analize zadovoljavaju empirijski definisane preporuke prema Wagner-u [18]. Za imperfekcije od 4,0 mm, što su ujedno i vrednosti imperfekcije koju preporučuje Evrokod za debljinu zida ljuske od 10,0 mm i klasu A proizvodne tolerancije, rezultati su nešto konzervativniji. Isto ponašanje uočeno je za veće vrednosti početnih imperfekcija. Rezlutati numeričke

The cylindrical shells made of S275 carbon steel exhibit relatively good agreement with modified curves by Wagner [18] for geometrical imperfections defined in accordance with the recommendations provided in EN 1993-1-6 [9] for the analysed shell length and radius. In the case of the shell 10,0 mm thick made of S275 carbon steel and the geometrical imperfections from 1,0 to 3,0 mm, the results of the numerical analysis satisfy empirically defined recommendations according to Wagner [18]. For imperfections of 4,0 mm, which are simultaneously the imperfection values recommended by the Eurocode for the shell wall thickness of 10,0 mm and class A of fabrication tolerance, the results are slightly more conservative. The same behaviour is observed for analize za cilindričnu ljusku od nerđajućeg čelika takođe su konzervativni u poređenju s preporukama prikazanim na slici 10b, čak i za najmanje vrednosti imperfekcija od 1,0 i 2,0 mm. Ovakav rezultat je očekivan, imajući u vidu to što su empirijske krive za redukcioni faktor, prikazane na slici 10, razvijene na osnovu rezultata eksperimentalnih ispitivanja cilindričnih ljuski od ugljeničnog čelika.

5 ZAKLJUČAK

Na osnovu rezultata numeričke analize, prikazanih u ovom radu, mogu se izvesti sledeći zaključci:

– Početne geometrijske imperfekcije imaju velik uticaj na nosivost tankih kružnih cilindričnih i konusnih ljuski. Imperfekcija od 1,0 mm za elemente debljine 10 mm izaziva redukciju nosivosti od 12% odnosno 17%, za ugljenični čelik i nerđajući čelik kod cilindričnih ljuski, respektivno. Povećanjem početnih geometrijskih imperfekcija do 5,0 mm, redukcija nosivosti dostiže i do 45% kod ugljeničnih čelika, odnosno 50% kod nerđajućeg čelika. Ova redukcija nosivosti kod konusnih ljuski od ugljeničnog čelika dostiže 30% u slučaju imperfekcija od 5 mm.

– Cilindrične ljuske od nerđajućeg čelika pokazuju progresivni pad tangentnog modula elastičnosti E_t, u oblasti napona između napona proporcionalnosti f_p i konvencionalne granice razvlačenja f_{02} u poređenju s ljuskama od ugljeničnog čelika, što utiče na znatno smanjenje krutosti.

– Povećanjem debljine zida cilindrične ljuske, smanjuje se uticaj početnih geometrijskih imperfekcija na nosivost elementa, a povećava uticaj materijalne nelinearnosti.

– Vrednost redukcionog (*KDF*) faktora za imperfekcije definisane u skladu s preporukama EN 1993-1-6 [9] za cilindrične ljuske od ugljeničnog čelika pokazale su dobro slaganje s preporukama za KDF faktora prema Wagner-u [18].

Kako je redukcija teorijske vrednosti kritičnog napona – koju propisuju standardi – velika, buduće unapređivanje proračunskih preporuka bazirano je na optimizaciji redukcionog faktora. Jedan od načina jeste formiranje i primena baze podataka o imperfekcijama kružnih cilindričnih ljuski, na osnovu sprovedenih eksperimentalnih ispitivanja (*Imperfection Data Bank*). Time bi se omogućila optimizacija proračuna i projektovanje za određeni tip cilindra, analizirajući samo rezultate dobijene na sličnim tipovima. Pored navedenog, nameće se pitanje može li se efekat početnih imperfekcija na smanjenje nosivosti izbeći odgovarajućim dizajnom, odnosno proizvodnim kvalitetom. Na ovom polju postoje mnogi radovi, a svi oni pokušavaju da dođu do ljuski neosetljivih na početne deformacije. Može se zaključiti da je polje analize ponašanja kružnih cilindričnih i konusnih ljuski i dalje umnogome otvoreno za istraživanje.

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the higher values of initial imperfections. The results of the numerical analysis for cylindrical shell of stainless steel are also conservative in respect to the recommendations displayed in figure 10b, even for the lowest values of imperfections of 1,0 and 2,0 mm. Such a result is expected regarding that the empirical curves for the knockdown factor given in figure 10 are developed based on the results of experimental tests of cylindrical shells made of carbon steel.

5 CONCLUSION

Based on the results of numerical analysis presented in this paper, the following conclusions can be drawn:

– Initial geometrical imperfections have a great impact on the buckling resistance of thin-walled cylindrical and conical shells. An imperfection of 1,0 mm causes the buckling resistance reduction of 12%, i.e. 17%, for carbon steel and stainless steel cylindrical shells, respectively. By increasing initial geometric imperfections up to 5.0 mm, the reduction of buckling resistance reaches up to 45% for carbon steel and 50% for stainless steel. This reduction in the case of conical, carbon steel shells reaches 30% for imperfection of 5mm.

– Cylindrical shells of stainless steel shows a progressive decline of tangent elastic modulus E_t, in the stress range between the proportionality limit stress f_p and the 0.2% proof stress f_{02} in comparison with carbon steel cylindrical shells, which causes the considerable decrease of stiffness.

– With the increase of the cylindrical shell thickness, the impact of initial geometrical imperfections on the buckling resistance is decreasing, and the impact of material nonlinearity is increasing.

– The value of KDF factor for geometrical imperfections determined according to the recommendations given in EN 1993-1-6 [9] for cylindrical shells made of carbon steel represent a good agreement with recommendations given by Wagner [18].

As the reduction of the theoretical value of critical stress, prescribed by the standards is high, the future improvement of design recommendations is based on the optimization of the knockdown factor. One way is forming and implementation of a data base on imperfections of cylindrical shells based on the conducted experimental tests (*Imperfection Data Bank*). This facilitates optimization of calculation and design of a specific type of cylinder, by analysing the result obtained on the similar types of cylinders. In addition to the previous, there is also a question whether the initial imperfections effect, resulting in reduced bearing capacity, can be avoided using appropriate design, i.e. production quality. There are many papers on this topic, and all of them are striving to attain the shells insensitive to initial deformations. Therefore, it can be concluded that the field of analysis of cylindrical and conical shells behaviour is still largely open to research.

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REZIME

POREĐENJE PONAŠANJA TANKIH CILINDRIČNIH I KONUSNIH LJUSKI OD UGLJENIČNOG I NERĐAJUĆEG ČELIKA

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Tanke kružne cilindrične i konusne ljuske predstavljaju jedan od složenijih konstruktivnih elemenata u pogledu ponašanja i osetljivosti na izbočavanje. U radu je dat kratak teorijski osvrt, s prikazom različitih, trenutno dostupnih, proračunskih preporuka. Prikazana je numerička analiza uticaja početnih imperfekcija na nelinearno ponašanje kružnih cilindričnih i konusnih ljuski. Analizirane su ljuske različite debljine zida, s konstantnim vrednostima dužine i prečnika ljuske, kao i s različitim vrednostima početnih imperfekcija. Analiza obuhvata uticaj materijalne i geometrijske nelinearnosti na ponašanje ljuski od ugljeničnog i nerđajućeg čelika, uključujući eksperimentalne krive napon-dilatacija. U radu je pokazano da materijalna nelineranost i početna geometrijska imperfekcija dovode do značajnog pada nosivosti na izbočavanje ljuski.

Ključne reči: ljuske srednje dužine, postkritično ponašanje, početne imperfekcije, izbočavanje, redukcioni faktor

SUMMARY

BEHAVIOR OF THIN-WALLED CYLINDRICAL AND CONICAL SHELLS - CARBON STEEL vs. STAINLESS STEEL

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Thin-walled cylindrical and conical shells represent one of the most complex structural elements considering their behaviour and susceptibility to buckling. A brief theoretical review including the presentation of different currently available design recommendations is given in this paper. Influence of initial imperfections on nonlinear behaviour of cylindrical and conical shells is also presented through numerical analysis. Shells with different wall thicknesses and different values of initial imperfections, but constant length and diameter of shell are analysed. Numerical analysis includes materially and geometrically nonlinear analysis of cylindrical and conical shells, using experimentally obtained stress-strain relation of carbon steel and stainless steel. Material nonlinearity and initial geometrical imperfections resulted in significantly lower buckling resistance of shells.

Key words: medium-length shells, post critical behaviour, initial imperfections, buckling, knockdown factor

ISPITIVANJE INTEGRITETA I NOSIVOSTI ŠIPOVA: METODOLOGIJA I KLASIFIKACIJA

PILE INTEGRITY AND LOAD TESTING: METHODOLOGY AND CLASSIFICATION

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1 UVOD

Fundiranje objekata na šipovima deo je kompleksne geotehničke problematike koja se u poslednjih nekoliko decenija intenzivno razvija u saradanji s drugim naučnim disciplinama na poljima: laboratorijskih analiza tla, ispitivanja tla putem *in-situ* testova, kao i numeričkim analizama, projektovanja prema propisima, tehnologiji izvođenja i slično. S obzirom na to što stepen nepouzdanosti parametara u geotehnici može biti znatno visok, u poređenju na, na primer, sa stepenom nepouzdanosti parametara kod konstrukcija, a imajući u vidu i to da broj ovih parametara kojima se modelira konstitutivni model ponašanja tla može biti znatan, može se pojaviti situacija vidnijeg odstupanja ponašanja projektovanog matematičkog modela u odnosu na izvedeno projektno rešenje šipova. Efekti ovih odstupanja iskazuju se u formi značajnijih sleganja konstrukcija i gubitka geotehničke nosivosti, odnosno sloma u tlu, pa i kolapsa konstrukcija. U tom smislu, da bi se maksimalno eliminisala razlika između ponašanja projektnog matematičkog modela i izvedenog projektnog rešenja šipova, razvijen je niz metoda ispitivanja šipova na mestu izgradnje objekta. Ekspanzija softversko-hardverskog inženjerstva, u poslednje dve decenije, omogućila je uvođenje multidisciplinarnog pristupa u analizi stanja šipova, gde se, u najvećem broju slučajeva, sprovode ispitivanja integriteta i nosivosti šipova. Gotovo svi testovi za ispitivanje integriteta i nosivosti šipova jesu *in-situ* elektronski instrumentalizovani, tako da se u *real time* ili naknadnim

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1 INTRODUCTION

Pile foundations for structures is a part of a complex geotechnical issue, which has been developed over the past few decades in cooperation with other scientific disciplines in the fields of: laboratory soil analysis, soil investigations via in-situ tests, numerical analysis, planning according to regulations, execution technology, and the like. Given that the unreliability level of the parameters in geotechnics may be quite high, compared to, for example, the level of unreliability in the case of constructions, and also that the number of these parameters which are used for modelling the constitutive model of soil behaviour can be significant, all of it may lead to a situation of considerable deviation in behaviour of mathematical model design from the executed pile solution design. The effects of these deviations are expressed in the form of quite significant construction settlement and geotechnical load capacity loss, i.e. soil failure, and even constructions collapse. Thus, a number of methods for pile testing at the construction site has been developed with the aim to maximally eliminate the difference between the behaviour of mathematical model design and executed pile design solution. The expansion of software-hardware engineering, in the last two decaes, has facilitated the introduction of multidisciplinary approach to pile condition analysis, out of which, in most cases, pile integrity and load tests are conducted. Almost all pile integrity and load tests are in-situ electronically instrumentalised, so the results relevant for

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procesiranjem podataka dobijaju rezultati od interesa za građevinsku praksu. Procedure sprovođenja testova integriteta i nosivosti šipova definisane su standardima, u kojima su prikazani elementi, kriterijumi i tokovi ispitivanja. Jedni od najdetaljnijih i najpouzdanijih standarda jesu američki ASTM standardi, gde postoje jasno definisane procedure, pravila i sistem kvaliteta ispitivanja. Ispitivanja integriteta šipova prema američkim standardima definisana su putem: ASTM D7949 [5], ASTM D5882 [6], ASTM D6760 [7], dok su ispitivanja nosivosti šipova definisana putem: ASTM D7383 [1], ASTM D1143 [2], ASTM D3689 [3], ASTM D3966 [4], ASTM D4945 [8], ASTM D8169 [9]. Takođe, evropski standard EN 1997-1:2004 [20], u određenom obimu, ima definisanu regulativu ispitivanja šipova, s tim što se ovi propisi umnogome odnose na međunarodne ISO standarde.

U modernom građevinarstvu, procedura izgradnje objekta fundiranog na šipovima sprovodi se u nekoliko koraka, pri čemu ispitivanje integriteta i nosivosti šipova predstavlja jednako bitnu kariku u kompletnom procesu. Na slici 1 prikazan je dijagram toka metodologije:

– ispitivanja tla (laboratorijska ispitivanja tla i *in-situ* ispitivanja tla, geotehnički elaborat),

– projektovanje objekta (projektno-tehnička dokumentacija),

– izgradnja objekta (projekat tehnologije gradnje objekta, izgradnja konstrukcije, dokument o tehnologiji izgradnje šipova, izgradnja šipova),

– ispitivanje konstrukcije (dokument o tehnologiji ispitivanja konstrukcije, izveštaj ispitivanja konstrukcije),

– ispitivanje šipova (dokument o tehnologiji ispitivanja šipova, izveštaj ispitivanja šipova).

Svaka faza, od ispitivanja tla, preko projektovanja i izgradnje, pa sve do ispitivanja konstrukcije i šipova, usmerena je procedurama definisanim u standardima o: ispitivanju tla, projektovanju objekata, izgradnji objekata, ispitivanju konstrukcija i ispitivanju šipova.

Postojeći inostrani standardi definišu smernice za ispitivanje šipova, međutim, postoje određeni elementi koje je potrebno dodatno unaprediti. U tom smislu, u ovom radu prikazani su neki segmenti unapređivanja postojećih standarda i sopstvene definicije određenih ključnih elemenata, koji su, između ostalog, razmatrani u istraživanjima [10], [11], [12], [13], [14], [15], [16], [17], [18], [19], [23], [24], [25], [26], [27], [28], [29]. Značajan doprinos u izučavanju i poboljšanju ispitivanja šipova prikazan je u radu [22], čije je izučavanje dodatno doprinelo razumevanju ove kompleksne problematike. Unapređivanje segmenata postojećih standarda ogleda se u: detaljnijem razjašnjenju pojedinih faza ispitivanja, te redukciji i selekciji metoda i postupaka ispitivanja u okviru jednog testa, a s obzirom na to što u postojećim standardima ispitivanja šipova postoji veći broj opcija ispitivanja koje se najčešće i ne koriste, prikazani su ključni elementi ispitivanja integriteta i nosivosti šipova, bez dodatnih (nepotrebnih) opcija koje, u najvećoj meri, zbunjuju investitora i nadzora prilikom samog sprovođenja ispitivanja šipova.

the practice of civil engineering are obtained in real time or through additional data processing. The procedures for conducting pile integrity and load tests are defined by standards, which describe elements, criteria, and testing processes. Among the most detailed and reliable standards are the USA ASTM standards, which have clearly defined procedures, rules, and quality test system. According to the USA standards, pile integrity tests are defined through: ASTM D7949 [5], ASTM D5882 [6], ASTM D6760 [7], whereas pile load tests are defined through: ASTM D7383 [1], ASTM D1143 [2], ASTM D3689 [3], ASTM D3966 [4], ASTM D4945 [8], ASTM D8169 [9]. In addition, European standard EN 1997-1:2004 [20] has a defined pile testing regulations to a certain extent, but they mostly refer to international ISO standards.

In modern civil engineering the procedure of building an object founded on piles is conducted in several steps, and in which case the pile integrity and load testing makes an equally important component in the whole process. The figure 1 shows a methodology flowchart:

– geotehnical investigation (laboratory soil testing and in-situ soil testing, geotechnical study),

– building design (technical-design documentation),

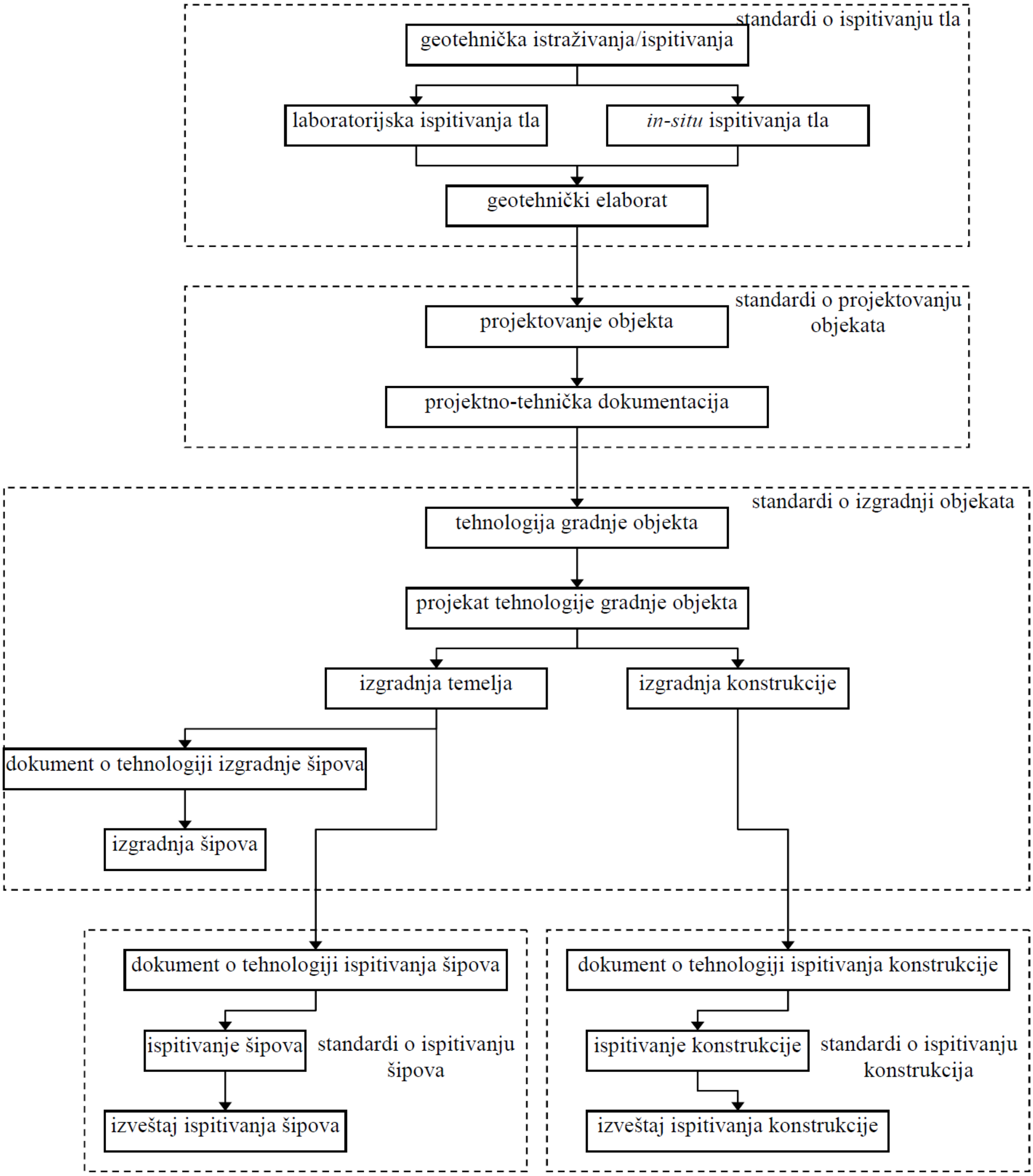
– building construction (construction technology project, building construction, document regarding the pile construction technology, pile construction),

– structural testing (document regarding the structural testing technology, structural testing report),

– pile testing (document regarding the pile testing technology, pile testing report).

Each phase, from soil testing, through designing and constructing, all the way to the structural and pile testing, is guided by the procedures defined in the standards of: geotehnical investigation, building design, building construction, structural testing and pile testing.

The existing foreign standards define the guidelines for pile testing, although there are certain elements, which need further improvement. In that regard, this paper presents certain segments of the improvement of the existing standards and our own definitions of certain key elements, which, among other things, have been examined in the testing [10], [11], [12], [13], [14], [15], [16], [17], [18], [19], [23], [24], [25], [26], [27], [28], [29]. A significant contribution to the study and pile testing improvement has been presented in the paper [22], and the study of which has further contributed to the understanding of this complex issue. The improvement of segments of the existing standards reflects in: a detailed explanation of individual testing phases, reduction and selection of testing methods and steps within one test, since in the existing pile testing standards there is a larger number of testing options which are not even used ordinarily. The key elements of pile integrity and load testing have been presented, without the additional options which, mostly, confuse the investor and supervisor during the pile testing conduction.



Slika 1. Dijagram toka metodologije: ispitivanja tla, projektovanja objekta, izgradnje objekta, ispitivanja konstrukcije i ispitivanja šipova

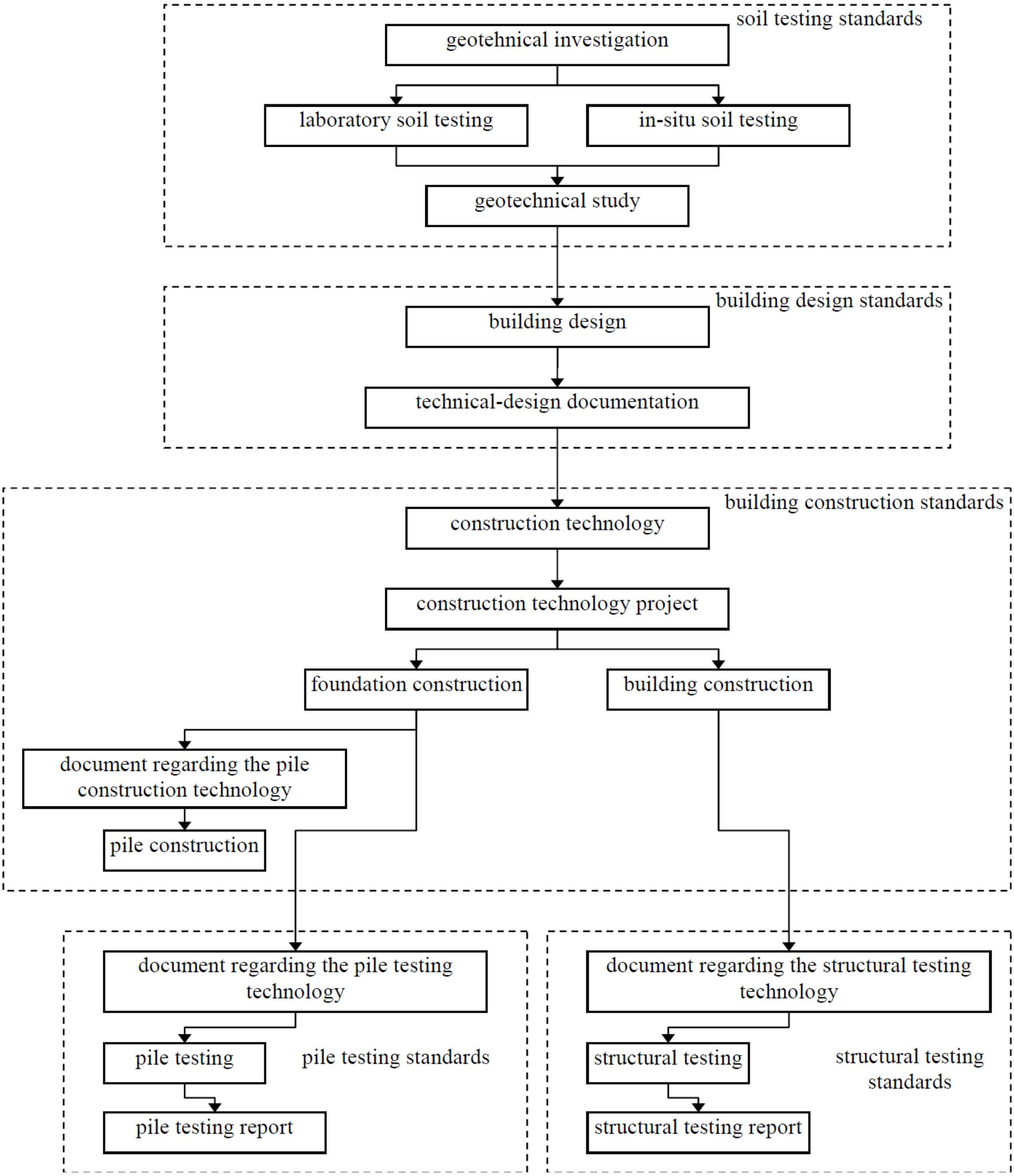


Figure 1. Methodology flowchart: soil testing, building design, building construction, structural testing, and pile testing

2 ISPITIVANJE INTEGRITETA ŠIPOVA

S obzirom na to što je razvijen veći broj testova ispitivanja integriteta šipova, ovde je prikazana sprovedena klasifikacija na:

2 PILE INTEGRITY TESTING

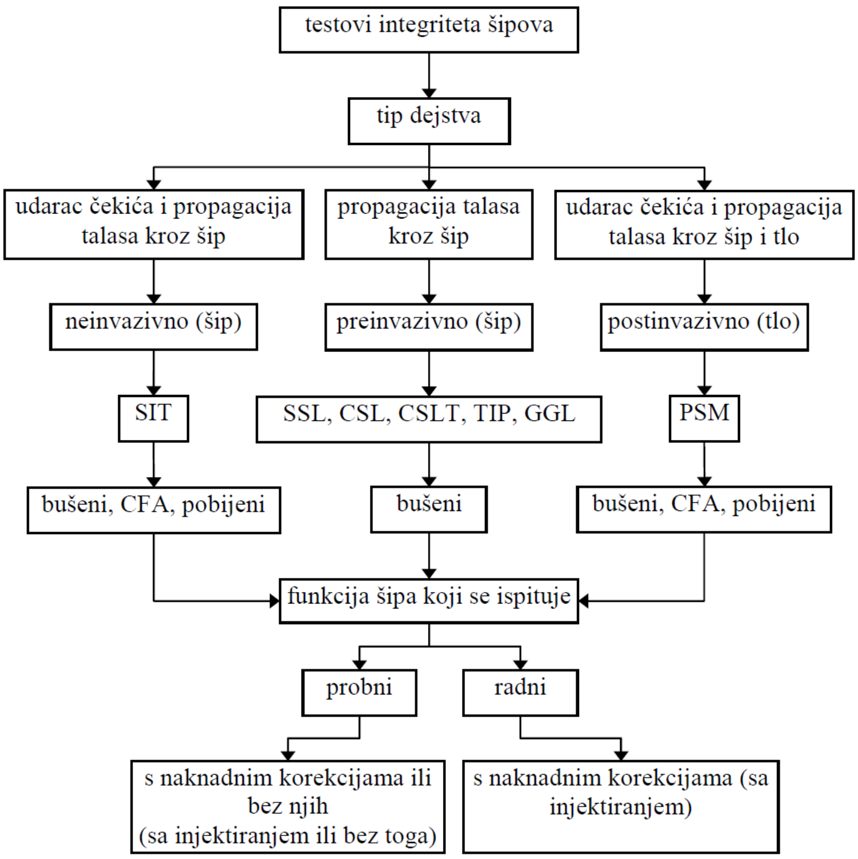
Since a larger number of pile integrity tests has been developed, the conducted classification is presented as the following:

– test integriteta šipa sa senzorom (SIT),
– test integriteta šipa sa jednom sondom (SSL),
– test integriteta šipa sa dve ili više sondi (CSL),
– test integriteta šipa sa 3D tomografskim prikazom (CSLT),
– test termalnog integriteta šipa (TIP),
– gama-gama test integriteta šipa (GGL),
– paralelna seizmička metoda (PSM).

Testovi integriteta šipova sprovode se kod: bušenih šipova izgrađenih tehnologijom bušenja i vađenja tla na površinu terena, šipova izgrađenih tehnologijom nevibriranja, odnosno bušenjem bez vađenja tla na površinu terena (CFA) i šipova izgrađenih tehnologijom pobijanja. Tipovi integriteta koji se ispituju ovim testovima klasifikovani su u nekoliko grupa: stvarna dužina šipa, promena porečnog preseka duž omotača šipa, pojava diskontinuiteta (prslina) duž omotača šipa, analiza defekata duž omotača šipa, analiza brzine propagacije talasa kroz šip, analiza gustine ili poroznosti betona i analiza temperature hidratacije cementa. U zavisnosti od samog pristupa šipu, ispitivanje integriteta može se sprovesti: preko glave šipa (neinvazivni test SIT), kroz instalirane cevi u šipu (preinvazivna priprema testa SSL, CSL, CSLT, TIP, GGL) i preko glave šipa i kroz instaliranu cev u tlu (neinvazivni test šipa, ali postinvazivna priprema tla PSM). Na slici 2 prikazan je dijagram toka opšte klasifikacije tipova testova integriteta šipova.

– SIT - *Sonic Integrity Test*,
– SSL - *Singlehole Sonic Logging*,
– CSL - *Crosshole Sonic Logging*,
– CSLT - *Crosshole Sonic Logging Tomography*,
– TIP - *Thermal Integrity Profiler*,
– GGL - *Gamma-Gamma Logging*,
– PSM - *Parallel Seismic Method*.

Pile integrity tests are carried out in the case of: bored piles built through the technology of drilling and extracting soil out onto the surface of the field, pile built through a non-vibration technology, i.e. by continuous flight auger (CFA), and the pile built through the driving technology. The integrity types examined in these tests are classified into several groups: the actual pile length, changes in the cross-section along the pile shell, the occurrence of discontinuity (cracks) along the pile shell, the analysis of the defects along the pile shell, the analysis of the velocity of the wave propagation through the pile, concrete density or porosity analysis, and the cement hydration temperature analysis. Depending on the available access to the pile, the integrity testing can be conducted: through the pile head (a non-invasive test SIT), through the tubes installed in the pile (a pre-invasive test preparation SSL, CSL, CSLT, TIP, GGL) and through the pile head and via the tube installed in the ground (non-invasive pile test, yet a post-invasive ground preparation PSM). Figure 2 shows a flowchart of the general pile integrity test types classification.



Slika 2. Dijagram toka opšte klasifikacije tipova testova integriteta šipova

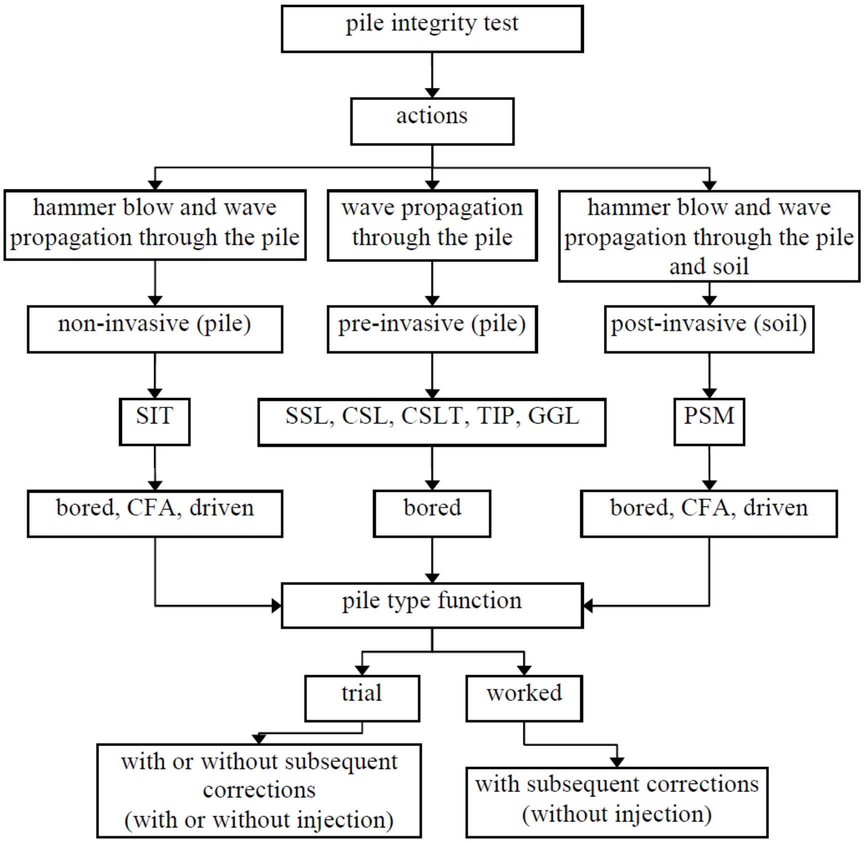


Figure 2. Flowchart of the general pile integrity test types classification

Test integriteta šipa sa senzorom (SIT ili PIT) u praksi se zove i test eha zvuka (SET) ili test eha šipa (PET), a pripada grupi niskodilatacionih testova (LST) [6]. Test integriteta šipa sa senzorom (SIT) zasniva se na teoriji propagacije talasa kroz šip, s ciljem utvrđivanja defekata/diskontinuiteta i dužine šipa. Ovaj test realizuje se na principu indukovanja, propagacije, refleksije, refrakcije i recepcije talasa u šipu. Indukcija talasa inicira se putem spoljašnjeg dejstva, udarom čekića, tako da je spoljašnji transmitovani signal, u opštem slučaju, impulsnog karaktera. Propagacija talasa kroz šip sprovodi se nakon iniciranja talasa, od glave do baze šipa i suprotno. Efekat refleksije javlja se na mestu kontakta dva različita medijuma, u konkretnom slučaju, na mestu baze šipa i tla, gde talas propagira ka glavi šipa. Refrakcija talasa jeste efekat prelamanja talasa na kontaktu dva medijuma, kao što je kontakt baze i omotača šipa s tlom. Bazni uslov, koji treba da je ispunjen kako bi se mogao efikasno razmatrati integritet šipa ovim testom, jeste da dužina talasa, koji se inicira, bude veća od prečnika šipa, tako da se propagacija talasa u šipu može razmatrati primenom jednodimenzionalne teorije rasprostiranja talasa u čvrstom medijumu. Zapis promene brzine propagacije talasa u vremenu, mereno na mestu glave šipa, prikazuje se putem reflektograma. Generalno razmatrajući, promene na reflektogramu javljaju se usled promene u bazi šipa, promene u prečniku duž omotača šipa, delimičnom inkluzijom tla u domen šipa, prslinama (većim, značajnijim), varijacijom kvaliteta materijala šipa, varijacijom slojeva tla i uticajem armaturnog čelika u šipu (jako

Sonic Integrity Test (SIT) or Pile Integrity Test (PIT), is also commonly known as Sonic Echo Test (SET) or Pile Echo Test (PET), and it is one of the Low Strain Tests (LST) [6]. SIT is based on the theory of wave propagation through the pile with the aim of determining defects/discontinuities and the length of the pile. This test is carried out on the principle of induction, propagation, reflection, refraction, and reception of the waves in the pile. The wave induction is initiated via the outside action, by a hammer hit, so the external transmitted signal is of impulse type, in general. Wave propagation through the pile is conducted after the wave induction, from the pile head to its base and vice versa. The reflection effect occurs where two different mediums come into contact, in this case, on the spot of pile base and the ground, where the wave propagates towards the pile head. Wave refraction is the effect of wave fraction on the point of contact between the two mediums, such as the contact between the pile base and shell with the ground. The basic condition, which needs to be fulfilled, so as to efficiently examine the pile integrity via this test, is that the length of the wave, which is initiated, is bigger than the pile diameter, so that the wave propagation in the pile can be examined through the application of the one-dimensional theory of wave propagation in a solid medium. The record of the wave propagation velocity in the period of time, measured on the pile head point, is presented via reflectogram. In general, the changes on the reflectogram occur due to the change in the pile base, the change in the diameter along the pile shaft, partial inclusion of the ground in the pile domain, cracks armirani šip). Test integriteta šipa sa senzorom (SIT) zasniva se na: dinamici kretanja krutog tela, talasnoj teoriji, metodi karakteristika, teoriji elastičnosti, dinamici konstrukcija, interakciji konstrukcija-tlo i teoriji i obradi signala. Prema dinamici kretanja krutog tela, razmatra se apliciranje spoljašnjeg dejstva udarom idealno krutog tela (čekića) o glavu šipa. Prema talasnoj teoriji, razmatraju se aspekti propagacije talasa kroz šip i tlo. Prema metodi karakteristika, razmatraju se aspekti kretanja odlazećih i dolazećih talasa sa identifikacijom defekata i diskontinuiteta u šipu. Prema teoriji elastičnosti, uzima se u obzir to što je konstitutivni model ponašanja šipa i tla linearno-elastičan. Prema dinamici konstrukcija, razmatraju se oscilacije šipa u interakciji sa tlom u vremenskom domenu. Prema interakciji konstrukcija-tlo, razmatra se spregnut problem statičke i dinamičke interakcije i reakcije dva medijuma (šip i tlo), bitno različitih fizičko-mehaničkih karakteristika. Prema teoriji i obradi signala, razmatraju se digitalizacija i procesiranje signala, s ciljem dobijanja odgovarajućih konačnih rezultata, primenljivih u građevinskoj inženjerskoj praksi, pomoću kojih se donose odluke o stanju integriteta šipa. Oprema za sprovođenje testa integriteta šipa sa senzorom (SIT) sastoji se iz: klasičnog mehaničkog čekića ili mehaničkog čekića povezanog električnim kablom za merenje karakteristika indukovanog signala, čije vreme trajanja ne sme biti duže od 1ms i ne sme prouzrokovati defekat šipa (za šipove prečnika manjeg od 1m koriste se čekići težine 0.5kg, 1.5kg i 3.5kg, a za šipove prečnika većeg od 1m koriste se čekići težine 1.5kg, 3.5kg i 6kg), senzora (akcelerometra) za prijem signala (opsega akceleracija minimalno 50g, rezonantne frekvencije minimalno 30kHz i tačnosti minimalno 5%) i uređaja za akviziciju, memorisanje, obradu i vizuelizaciju podataka. Test integriteta šipa sa senzorom (SIT) sprovodi se u nekoliko koraka: analiziraju se svi relevantni podaci u pasošu šipa i geotehničkom elaboratu, setuje se oprema, podaci i parametri za šip koji se ispituje, aplicira se udarac čekića u zoni centra ose šipa, na prethodno pripremljenu površinu glave šipa se postavi senzor (akcelerometar), pri čemu treba da je udaljenost pozicije senzora od mesta aplikacije udarca maksimalno 300mm, postupak se ponavlja nekoliko puta (minimalno 3 puta), a ukoliko se pokaže potrebnim koriguju se parametri skaliranja i filtriranja (za šipove prečnika većeg od 500mm primenjuju se 4 merenja - u 2 ortogonalna-radijalna pravca), konstruiše se reprezentativni reflektogram za prosečne vrednosti (3 reflektograma) i naknadno se sprovodi obrada podataka dobijenih reflektograma.

Test integriteta šipa sa sondama (CSL) pripada grupi niskodilatacionih testova (LST), pri čemu postoje varijantna rešenja s jednom sondom ili većim brojem sondi [7]. Test integriteta šipa ili test eha zvuka s jednom sondom (SSL ili SHUT) sprovodi se radi analize defekata/diskontinuiteta u šipu, a zasniva se na propagaciji talasa primenom sonde u kojoj su smešteni transmiter i risiver. Talasi se emituju putem transmitera, propagiraju kroz vodu, zid cevi, šip, reflektuju na mestima kontakta s tlom i prihvataju risiverom. S

(bigger, more significant), pile material quality variation, soil layers' variation, and the influence of the reinforcing steel in the pile (high reinforced pile). SIT is based on: rigid-body dynamics, wave theory, method of characteristics, theory of elasticity, dynamics of structures, soil-structure interaction (SSI), and theory and signal processing. According to the rigid-body dynamics, the application of an external action through a hit from an ideally rigid-body (hammer) onto the pile head is considered. According to the wave theory, the aspects of wave propagation through the pile and the soil are considered. According to the method of characteristics, the propagation aspects of downward and upward waves with the identification of defects and discontinuities in the pile are considered. According to the theory of elasticity, we take into account the fact that the constitutive models of the pile and soil are linear-elastic. According to the dynamics of structures, the pile oscillations during the interaction with the soil within the time domain are considered. According to soil-structure interaction, the coupled problem of static and dynamic interaction and reaction of two mediums (pile and soil), with significantly different physical-mechanical properties are considered. According to the theory and signal processing, signal digitalisation and processing are considered with the aim of acquiring corresponding final results, practically applicable in civil engineering, and with the help of which the decisions regarding the pile integrity conditions are reached. SIT equipment consists of: a conventional mechanic hammer or a mechanic hammer connected to an electric cable for measuring induced signal characteristics, whose period must not be longer than 1ms and must not cause a defect in the pile (for piles of less than 1m in diameter, hammers weighing 0.5kg, 1.5kg and 3.5kg are used, and for piles bigger than 1m in diameter, hammers weighing 1.5kg, 3.5kg and 6kg are used), sensor (accelerometer) for signal reception (of minimum acceleration range 50g, minimum resonance frequencies 30kHz and minimum accuracy 5%), and data acquisition, storage, processing, and visualisation equipment. SIT is conducted in a few steps: all the relevant data in the pile's passport and geotechnical study are analyzed, the equipment, data, and parameters are set for the pile being tested, hammer hit is applied in the pile axis centre zone, a sensor (accelerometer) is placed onto the previously prepared pile head surface, while the sensor position should not be more than 300mm away from the hit application point, the process is repeated several times (minimum of 3), and, if it turns out to be necessary, the scaling and filtering parameters are adjusted (4 measurements are applied for the piles bigger than 500mm in diameter - in 2 orthogonal radial directions), a representational reflectogram for average values (3 reflectograms) is produced, and data processing from acquired reflectograms is additionally conducted.

Crosshole Sonic Logging (CSL) belongs to the group of low strain tests (LST), while there are variant solutions with one or a number of probes [7]. Singlehole Sonic Logging (SSL) or Singlehole Ultrasonic Test (SHUT) is conducted with the aim of analyzing defects/discontinuities in the pile, and it is based on the wave propagation by implementation of a probe which contains a transmitter and a receiver. The waves are emitted via the transmitter, they propagate through the obzirom na to što se transmiter spušta/podiže vertikalno naniže/naviše duž šipa, kontinualno se prati signal koji se dobija putem risivera. U tom smislu, ovaj test najviše se koristi za direktnu analizu defekata u poprečnom preseku, pa se integracijom odgovora dobija kompletnija slika o stanju šipa. Primena testa eha zvuka, s jednom sondom (SSL), najzastupljenija je kod šipova manjeg prečnika ili kod šipova kod kojih se, usled odgovarajućih ograničenja za ispitivanje, može koristiti mala zona poprečnog preseka. Test integriteta šipa ili test eha zvuka s dve sonde ili više sondi (CSL ili CHUT), slično testu integriteta šipa s jednom sondom (SSL), zasniva se na propagaciji talasa primenom sondi, međutim, s razdvojenim transmiterom i risiverom. U jednu cev postavlja se transmiter, a u drugu risiver, tako da se postepenim spuštanjem/podizanjem transmitera i risivera vertikalno naniže/naviše prati stanje šipa po poprečnim presecima duž omotača šipa. Integracijom dobijenih analiza duž šipa, dobija se kompletna slika o mogućim defektima/diskontinuitetima, pri čemu se mogu detektovati i manji defekti, prsline (veće, značajnije), šupljine, intruzija sa vodom/tlom i betonska gnezda. Najveći broj defekata identifikuje se u okolini cevi, međutim, za veće prečnike šipova može se koristiti veći broj cevi za sprovođenje testa. Na taj način, dobija se kvalitetnija slika stanja integriteta šipa. 2D ili 3D tomografija integriteta šipa (CSLT) jeste dodatno unapređena verzija testa integriteta šipa s dve sonde ili više sondi (CSL). Prilikom ovog testa, beleže se emitovani signali pod različitim uglovima i pravcima, a ne samo direktno horizontalno između transmitera i risivera, tako da se naknadnim procesiranjem i rekonstrukcijom dobija 2D i 3D prikaz stanja u šipu, pri čemu se vizuelno volumenski mogu jasno uočiti i izdvojiti defekti/diskontinuiteti kao nezavisne celine. Takođe, ovi defekti i kompletan šip mogu se prikazati u 4D. Test integriteta šipa sa sondama (CSL) zasniva se na: talasnoj teoriji, teoriji elastičnosti i teoriji i obradi signala. Prema talasnoj teoriji, razmatraju se aspekti propagacije talasa kroz šip, vodu i cevi. Prema teoriji elastičnosti, uzima se u obzir to što je konstitutivni model ponašanja šipa linearno-elastičan. Prema teoriji i obradi signala, razmatraju se digitalizacija i procesiranje signala, s ciljem dobijanja odgovarajućih konačnih rezultata primenljivih u građevinskoj inženjerskoj praksi, pomoću kojih se donose odluke o stanju integriteta šipa. Oprema za sprovođenje testa integriteta šipa sa sondama (CSL) sastoji se iz: metra s tegom za preliminarnu proveru dužine i nezapušenosti instaliranih cevi, transmitera (generiše ultrazvučni signal frekvencije od 30kHz do 100kHz), risivera (frekvencije do 50kHz), kablova za povezivanje transmitera i risivera sa uređajem za akviziciju podataka, tripoda za kablove sa senzorima za pozicioniranje sondi i uređaja za akviziciju, memorisanje, obradu i vizuelizaciju podataka. Test integriteta šipa sa sondama (CSL) sprovodi se u nekoliko koraka: analiziraju se svi relevantni podaci u pasošu šipa i geotehničkom elaboratu, definiše se orijentacija cevi u šipu, u odnosu na poziciju ostalih šipova i naglavnu ili temeljnu ploču, cevi se ispune vodom pre betoniranja šipa ili sat vremena nakon toga, setuju se oprema, podaci i parametri za šip koji se ispituje, u cev se paralelno spuštaju/podižu transmiter i risiver, a za SSL test transmiter pa risiver, mada su zajedno povezani istim kablom, transmiter i risiver se spuštaju približno do 50mm od kraja cevi, a zatim

water, wall, tubes, pile, are reflected on the points of contact with the soil, and picked up by the receiver. Since the transmitter is lowered/lifted vertically downwards/upwards along the pile, the signal, picked up through the receiver, is continuously being monitored. In that respect, this test is mostly used for a direct analysis of the defects in the cross-section, and the response integration gives a more complete image of the pile condition. Singlehole Sonic Logging (SSL) application is the most frequent in the case of piles of a smaller diameter or in the case of the piles in which, as a result of corresponding testing limitations, a small zone of the cross-section can be used. Crosshole Sonic Logging (CSL) or Crosshole Ultrasonic Test (CHUT), similarly to SSL, is based on wave propagation by probes implementation, though with a separate transmitter and a receiver. Transmitter is placed in one tube, and receiver in the other, so that through the gradual lowering/lifting of the transmitter and the receiver vertically downwards/upwards we can record the pile condition on the cross-sections along the pile shaft. Integration of the obtained analyses along the pile, gives a complete image of the possible defects/discontinuities, also allowing for detection of the smaller defects, cracks (larger, more significant), cavities, water/soil intrusion, and honeycombs in the concrete texture. The largest number of defects is identified in the tube adjacent area, while a larger number of tubes to perform the tests are used for larger pile diameters. Thus, a higher quality image of the pile condition is obtained. A 2D or 3D Crosshole Sonic Logging Tomography (CSLT) is an additionally improved version of the CSL. In this test, signals emitted under different angles and directions are recorded, not only directly horizontally between the transmitter and the receiver, hence, the subsequent processing and reconstruction gives 2D and 3D pictures of the condition in the pile, whereby the defects/discontinuities can clearly be visually, in full volume, spotted as independent units. Also, the defects and the whole pile can be represented in 4D. CSL is based on: wave theory, theory of elasticity, and theory and signal processing. According to the wave theory, the aspects of wave propagation through the pile, water, and tubes are considered. According to the theory of elasticity, we take into account that the constitutive model of the pile is linear-elastic. According to the theory and signal processing, signal digitalisation and processing are considered with the aim of acquiring corresponding final results, practically applicable in civil engineering, and with the help of which the decisions regarding the pile integrity conditions are reached. CSL equipment consists of: a meter with a weight for a preliminary check of the length and the potential blockages in the installed tubes, a transmitter (it generates an ultrasonic signal of 30kHz to 100kHz in frequency), a receiver (up to 50kHz frequency), cables for connecting the transmitter and the receiver with the data acquisition equipment, a tripod for cables with sensors for probes positioning, and data acquisition, storage, processing, and visualisation equipment. CSL is conducted in several steps: all the relevant data in the pile's passport and geotechnical study are analyzed. The tube orientation within the pile is defined in relation to the position of the other piles and the pile cap or the base foundation plate. The tubes are filled with water before or an hour after pouring concrete. The započinje proces snimanja signala uz postepeno (polako) podizanje transmitera i risivera, kada se ispitivanje sprovodi u cilju 2D ili 3D tomografskog prikaza integriteta šipa (CSLT), preporuka je da vertikalna visinska razlika između transmitera i risivera bude do 300mm (najviše do 500mm), konstruišu se adekvatni dijagrami, odnosno ultrazvučni profili i naknadno se sprovodi obrada podataka dobijenih ispitivanjem.

Test termalnog integriteta šipa (TIP) pripada grupi niskodilatacionih testova (LST) i zasniva se na analizi temperature hidratacije cementa radi identifikacije defekata/diskontinuiteta kod bušenih i nekih tipova CFA šipova [5]. Karakteristike ovog testa jesu to što se mogu analizirati kompletni poprečni preseci duž šipa i indicirati zone betona viših i nižih kvaliteta, a mogu se otkriti i veoma mali defekti/diskontinuiteti u šipu. Varijantna rešenja ovog testa integriteta su: s jednom sondom, dve sonde ili više sondi i kablovima s termalnim senzorima. Test termalnog integriteta šipa (TIP) zasniva se na: termodinamičkoj teoriji, teoriji elastičnosti i teoriji i obradi signala. Prema termodinamičkoj teoriji, razmatraju se aspekti provođenja toplote kroz šip. Prema teoriji elastičnosti, uzima se u obzir to što je konstitutivni model ponašanja šipa linearno-elastičan. Prema teoriji i obradi signala, razmatraju se digitalizacija i procesiranje signala, s ciljem dobijanja odgovarajućih konačnih rezultata primenljivih u građevinskoj inženjerskoj praksi, pomoću kojih se donose odluke o stanju integriteta šipa. Oprema za sprovođenje testa termalnog integriteta šipa (TIP) sastoji se iz: metra s tegom za preliminarnu proveru dužine i nezapušenosti instaliranih cevi (ukoliko se koristi sistem sa cevima), sonde (4 ortogonalno postavljena senzora po jednoj sondi, ukoliko se koristi sistem sa cevima), kablova s termalnim senzorima (ukoliko se koristi sistem sa senzorima), kablova za povezivanje sonde ili kablova i senzora sa uređajem za akviziciju podataka, tripoda za kablove sa senzorima za pozicioniranje sondi i uređaja za akviziciju (frekvencija semplovanja signala minimalno 1Hz), memorisanje, obradu i vizuelizaciju podataka. Test termalnog integriteta šipa (TIP) s cevima sprovodi se u nekoliko koraka: analiziraju se svi relevantni podaci u pasošu šipa i geotehničkom elaboratu, definiše se orijentacija cevi u šipu, u odnosu na poziciju ostalih šipova i naglavnu ili temeljnu ploču, setuje se oprema, podaci i parametri za šip koji se ispituje, u cev se spušta sonda do 10cm od kraja (dna) cevi, a zatim započinje očitavanje uz postepeno (polako) podizanje, konstruiše se dijagram promene temperature duž šipa i naknadno se sprovodi obrada podataka dobijenih ispitivanjem. Test termalnog integriteta šipa (TIP) s kablovima i termalnim senzorima sprovodi se u nekoliko koraka: analiziraju se svi relevantni podaci u pasošu šipa i geotehničkom elaboratu, prethodno instalirani kablovi i senzori povezuju se sa uređajem za akviziciju podataka, setuju se oprema, podaci i parametri za šip koji se ispituje, senzorima se detektuje promena toplote hidratacije cementa, konstruiše se dijagram promene temperature duž šipa i naknadno se sprovodi obrada podataka dobijenih ispitivanjem.

equipment, data, and parameters are set for the pile being tested. The transmitter and the receiver are lowered/lifted in the tube in parallel with each other. For SSL the transmitter goes first and then the receiver. Although they are connected via the same cable, the transmitter and the receiver are lowered down approximately to the last 50mm of the tube, and then begins the signal recording process with gradual (slow) transmitter and receiver lifting, when the examination is conducted for the purposes of 2D or 3D CSLT. It is recommended that the vertical difference in height beween the transmitter and the receiver is up to 300mm (maximum 500mm). Adequate diagrams are constructed, i.e. the ultrasonic profile, and the data acquired through testing are additionally processed.

Thermal Integrity Profiler (TIP) belongs to the group of low strain tests (LST) and it is based on the cement hydration temperature analysis, with the aim of identifying the defects/discontinuities in bored and some types of CFA piles [5]. This test is characterised by the possibility of analysing the whole cross-sections along the pile and indicating zones of high and low-quality concrete, as well as detecting even the very small defects/discontinuities in the pile. The variant solutions of this integrity test are with: one probe, two or more probes and cables with thermal sensors. TIP is based on: thermodynamic theory, theory of elasticity, and theory and signal processing. According to thermodynamic theory, the aspects of heat conduction through the pile are considered. According to the theory of elasticity, we take into account that the constitutive model of the pile is linear-elastic. According to the theory and signal processing, signal digitalisation and processing are considered with the aim of acquiring corresponding final results, practically applicable in civil engineering, and with the help of which the decisions regarding the pile integrity conditions are reached. TIP equipment consists of: a meter with a weight for a preliminary check of the length and the potential blockages in the installed tubes (if the system with tubes is used), probes (4 orthogonally oriented sensors per probe, if the system with tubes is used), cables with thermal sensors (if system with sensors is used), cables for connecting the probe or cables and sensors with data acquisition device, a tripod for cables with sensors for probes positioning, and data acquisition (signal sampling frequency of minimum 1Hz), storage, processing, and visualisation equipment. TIP with tubes is conducted in several steps: all the relevant data in the pile's passport and geotechnical study are analyzed, we define the tube orientation within the pile, relative to the position of the other piles and the pile cap or the base foundation plate. The equipment, data, and parameters are set for the pile being tested, a probe is lowered inside the tube down to 10 cm from the tube's end (bottom), and then the reading process begins with gradual (slow) lifting, a diagram of the temperature change along the pile is constructed, and the data acquired through testing are additionally processed. TIP with cables and thermal sensors is conducted in several steps: all the relevant data in the pile's passport and geotechnical study are analyzed, the previously installed cables and sensors are connected to the data acquisition device, the equipment, data, and parameters are set for the pile being tested, the change in the cement hydration heat is

Gama-gama test integriteta šipa (GGL) pripada grupi niskodilatacionih testova (LST) i zasniva se na emitovanju i propagaciji gama zraka kroz šip, analizirajući gustinu ili poroznost betona i identifikujući defekte/diskontinuitete [21]. Ovaj test sprovodi se tako što se kroz instalirane cevi (potpuno suve) vertikalno naniže/naviše spušta/podiže integrisana sonda transmiter-risiver ili nezavisne sonde transmiter i risiver. Na osnovu etalonske vrednosti gustine betona, za svaki korak merenja, utvrđuje se odstupanje i konstruiše dijagram s prikazom zona visokih gustina, zona niskih gustina i tranzicione zone. Gama-gama test integriteta šipa (GGL) zasniva se na: talasnoj teoriji, teoriji elastičnosti i teoriji i obradi signala. Prema talasnoj teoriji, razmatraju se aspekti propagacije gama zraka kroz šip i cevi. Prema teoriji elastičnosti, uzima se u obzir to što je konstitutivni model ponašanja šipa linearno-elastičan. Prema teoriji i obradi signala, razmatraju se digitalizacija i procesiranje signala, s u ciljem dobijanja odgovarajućih konačnih rezultata primenljivih u građevinskoj inženjerskoj praksi, pomoću kojih se donose odluke o stanju integriteta šipa. Oprema za sprovođenje gama-gama testa integriteta šipa (GGL) sastoji se iz: metra s tegom za preliminarnu proveru dužine i nezapušenosti instaliranih cevi, transmitera (izvor emitovanja gama zraka treba da je cezijum 137), risivera (detektor gama zraka treba da se zasniva na principu rada *Geiger-Mueller*-ovog brojača), kablova za povezivanje transmitera i risivera sa uređajem za akviziciju podataka, tripoda za kablove sa senzorima za pozicioniranje sondi i uređaja za akviziciju, memorisanje, obradu i vizuelizaciju podataka. Gama-gama test integriteta šipa (GGL) sprovodi se u nekoliko koraka: analiziraju se svi relevantni podaci u pasošu šipa i geotehničkom elaboratu, definiše se orijentacija cevi u šipu, u odnosu na poziciju ostalih šipova i naglavnu ili temeljnu ploču, setuju se oprema, podaci i parametri za šip koji se ispituje, u cev se spuštaju/podižu transmiter i risiver (transmiter je s donje, a risiver s gornje strane, pri čemu radijus detekcije gama zraka treba da je od minimalno 7.5cm do maksimalno 12cm) - ispitivanje s jednom cevi, transmiter i risiver se spuštaju do 15cm od kraja (dna) cevi, a zatim započinje proces očitavanja uz postepeno (polako) podizanje transmitera i risivera (interval očitavanja treba da je maksimalno 4cm, uz minimalnih 200 očitavanja za ceo šip) - ispitivanje s dve cevi ili više cevi, konstruišu se adekvatni dijagrami s prikazom zona visokih gustina, zona niskih gustina i tranzicione zone i naknadno se sprovodi obrada podataka dobijenih ispitivanjem.

Paralelna seizmička metoda (PSM) pripada grupi niskodilatacionih testova (LST), a ovim postupkom se indirektnim putem, propagacijom talasa kroz šip i tlo, utvrđuje postojanje defekata/diskontinuiteta u šipu, a takođe određuje se njegova dužina. Ovaj test je posebno koristan za određivanje dužine postojećih šipova objekata. Aplikacijom spoljašnjeg dejstva po glavi ili bočnoj strani glave šipa, stvaraju se vibracije koje se prenose kroz šip i tlo, gde se primenom risivera beleže signali u digitalnom formatu, koji se naknadno obrađuju.

detected through the sensors, a diagram of the temperature change along the pile is constructed, and the data acquired through testing are additionally processed.

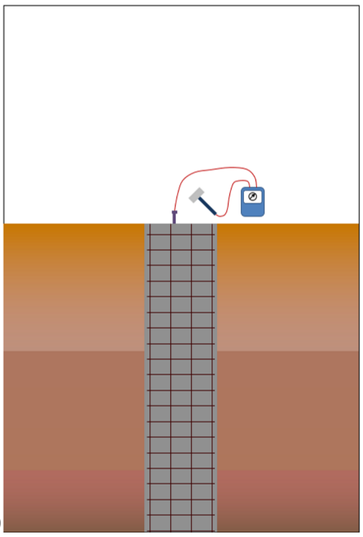
Gamma-Gamma Logging (GGL), belongs to the group of low strain tests (LST) and it is based on the emission and propagation of gamma rays through the pile, thus analysing the concrete density or porosity and identifying the defects/discontinuities [21]. This test is performed by lowering/lifting an integrated transmitter-receiver probe or independent transmitter and receiver probes vertically downward/upward through the installed tubes (completely dry). Based on the standard value of concrete density, for each step of the measuring, deviation is determined, and a diagram showing high density zones, low density zones and transition zones is constructed. GGL is based on: wave theory, theory of elasticity, theory and signal processing. According to the wave theory, we consider the aspects of the gamma ray propagation through the pile and tubes. According to the theory of elasticity, we take into account that the constitutive model of the pile is linear-elastic. According to the theory and signal processing, we consider signal digitalisation and processing with the aim of acquiring corresponding final results, practically applicable in civil engineering, and with the help of which the decisions regarding the pile integrity conditions are reached. GGL equipment consists of: a meter with a weight for a preliminary check of the length and the potential blockages in the installed tubes, a transmitter (gamma ray source should be caesium 137), a receiver (gamma ray detector should be based on the operation principle of the *Geiger-Mueller* counter), cables for connecting the transmitter and the receiver with the data acquisition device, a tripod for cables with sensors for probes positioning, and data acquisition, storage, processing, and visualisation equipment. GGL is conducted in several steps: all the relevant data in the pile's passport and geotechnical study are analyzed, then the tube orientation within the pile relative to the position of the other piles and the pile cap or the base foundation plate is defined, the equipment, data, and parameters are set for the pile being tested, the transmitter and the receiver are lowered/lifted within the tube (the transmitter is on the bottom side and the receiver on the top side, whereas the gamma ray detection radius should range between the minimum of 7.5cm to the maximum of 12cm) - the testing with one tube, the transmitter and the receiver are lowered down to 15cm from the tube's end (bottom), and then the reading process begins with gradual (slow) transmitter and receiver lifting (the reading interval should be up to the maximum of 4 cm, with a minimum of 200 readings for the whole pile) - the testing with two or more tubes, a diagram of the high density zones, low-density zones and transition zones is constructed, and the data acquired through testing are additionally processed.

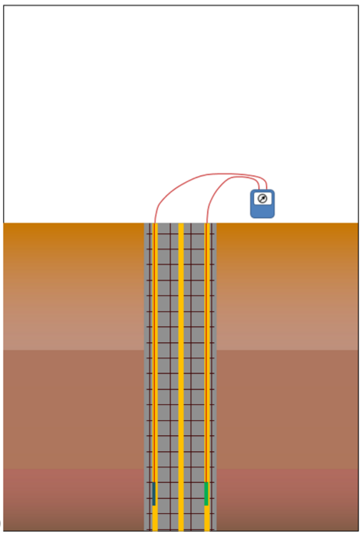
Parallel Seismic Method (PSM) belongs to the group of low strain tests (LST), and through this procedure, by wave propagation through the pile and the soil, we indirectly establish the existence of defects/discontinuities in the pile, as well as determine its length. This test is particularly useful for determining the depth of the existing piles. Application of an external action on the pile's head or its lateral side produces vibrations which are transmitted through the pile and the soil, Analizom serije zabeleženih signala, utvrđuju se stvarna dužina šipa i eventualno postojanje defekata/diskontinuiteta. Paralelna seizmička metoda (PSM) zasniva se na: talasnoj teoriji, teoriji elastičnosti i teoriji i obradi signala. Prema talasnoj teoriji, razmatraju se aspekti propagacije talasa kroz šip, tlo, cev i vodu. Prema teoriji elastičnosti, uzima se u obzir to što je konstitutivni model ponašanja šipa linearno-elastičan. Prema teoriji i obradi signala, razmatraju se digitalizacija i procesiranje signala u cilju dobijanja odgovarajućih konačnih rezultata primenljivih u građevinskoj inženjerskoj praksi, pomoću kojih se donose odluke o stanju integriteta šipa. Oprema za sprovođenje paralelne seizmičke metode (PSM) sastoji se iz: klasičnog mehaničkog čekića ili mehaničkog čekića povezanog električnim kablom za merenje karakteristika indukovanog signala, čije vreme trajanja ne sme biti duže od 1ms i ne sme prouzrokovati defekat šipa, sonde (hidrofona) za prijem signala (longitudinalnih i transverzalnih talasa) na svakih 10cm do 50cm duž šipa (frekvencija semplovanja minimalno 50kHz), čelične cevi i uređaja za akviziciju, memorisanje, obradu i vizuelizaciju podataka. Paralelna seizmička metoda (PSM) sprovodi se u nekoliko koraka: analiziraju se svi relevantni podaci u pasošu šipa i geotehničkom elaboratu, cev se utiskuje u tlo, a zatim ispuni vodom, setuju se oprema, podaci i parametri za šip koji se ispituje, u cev se spušta/podiže sonda (hidrofon) do kraja cevi, uz kontinualno snimanje signala i aplikaciju udarca čekićem, postupak se ponavlja nekoliko puta (minimalno 3 puta), a ukoliko se pokaže potrebnim koriguju se parametri skaliranja i filtriranja, konstruišu se reprezentativni signali u vremenskom domenu i naknadno se sprovodi obrada podataka dobijenih signala.

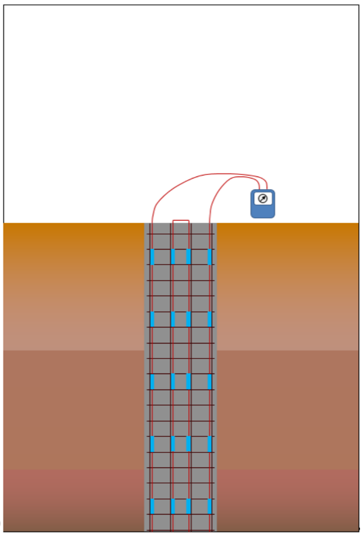
Na slici 3 dati su opšti šematski prikazi: a) testa integriteta šipa sa senzorom (SIT), b) testa integriteta šipa sa dve ili više sondi (CSL), c) testa termalnog integriteta šipa sa kablovima i termalnim senzorima (TIP), d) gama-gama testa integriteta šipa (GGL), e) testa integriteta šipa paralelnom seizmičkom metodom (PSM).

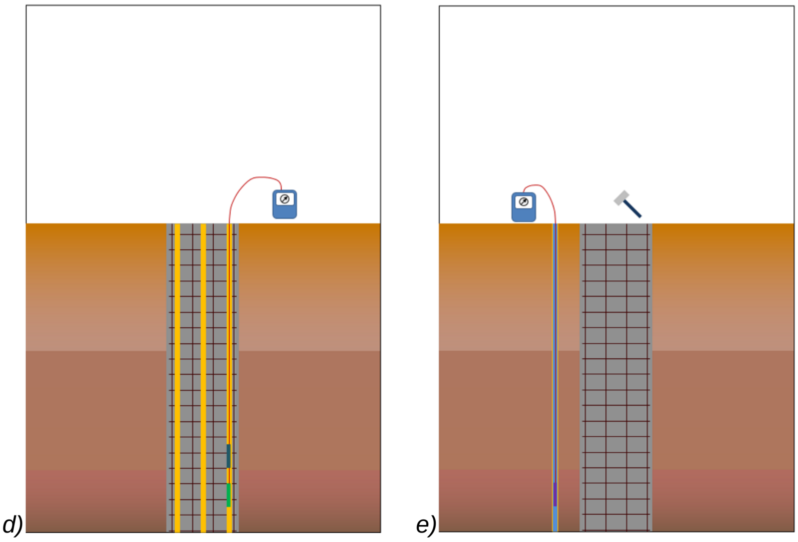
where the receiver records the signals in digital format, which are additionally processed. Through the analysis of the recorded signals we establish the actual length of the pile as well as the potential existence of any defects/discontinuities. PSM is based on: wave theory, theory of elasticity, theory and signal processing. According to the wave theory, the aspects of wave propagation through the pile, soil, tube and water are considered. According to the theory of elasticity, we take into account that the constitutive model of the pile is linear-elastic. According to the theory and signal processing, we consider signal digitalisation and processing with the aim of acquiring corresponding final results, practically applicable in civil engineering, and with the help of which the decisions regarding the pile integrity conditions are reached. PSM equipment consists of: a conventional mechanic hammer or a mechanic hammer connected to an electric cable for measuring induced signal characteristics, whose period must not be longer than 1ms and must not cause a defect in the pile, a probe (hydrophone) for signal reception (longitudinal and transverse waves) at every 10cm to 50cm along the pile (sampling frequency of minimum 50kHz), a steel tube, and data acquisition, storage, processing, and visualisation equipment. PSM is conducted in a few steps: all the relevant data in the pile's passport and geotechnical study are analysed, the tube is pressed into the ground, then filled with water, the equipment, data, and parameters are set for the pile being tested, a probe (hydrophone) is lowered/lifted inside the tube to the tube's end, while continuously recording the signal and applying the hammer hits, the procedure is repeated several times (at least 3 times), and if turns out to be necessary, the parameters of scaling and filtering are adjusted, representative signals are constructed in time domain, and the acquired signal data are subsequently processed.

Figure 3 gives a general scheme of: a) SIT, b) CSL, c) TIP with cables and thermal sensors, d) GGL, e) PSM.

a) 

b) 

c) 



Slika 3. Opšti šematski prikazi: a) testa integriteta šipa sa senzorom (SIT), b) testa integriteta šipa sa dve sonde ili više sondi (CSL), c) testa termalnog integriteta šipa sa kablovima i termalnim senzorima (TIP), d) gama-gama testa integriteta šipa (GGL), e) testa integriteta šipa paralelnom seizmičkom metodom (PSM)

Figure 3. The general scheme of: a) SIT, b) CSL, c) TIP with cables and thermal sensors, d) GGL, e) PSM

3 ISPITIVANJE NOSIVOSTI ŠIPOVA

S obzirom na to što je razvijen veći broj testova nosivosti šipova, ovde je prikazana sprovedena klasifikacija na:

- statički test opterećenja šipa (SLT),
- dinamički test opterećenja šipa (DLT),
- dinamički test opterećenja šipa pri pobijanju (PDT),
- hibridnamički test opterećenja šipa (HLT),
- statnamički test opterećenja šipa (SNLT),
- bidirekcioni statički test opterećenja šipa (BDSLT),
- test aksijalnog zatezanja šipa (ATT),
- test horizontalnog opterećenja šipa (LLT).

Tipovi nosivosti koje se ispituju ovim testovima klasifikovani su u tri grupe: aksijalni pritisak (kod svih testova ovog tipa SLT, DLT, PDT, HLT, SNLT aksijalna nosivost utvrđuje se opterećenjem šipa vertikalno naniže silom pritiska, dok se kod BDSLT šip opterećuje vertikalno naniže i vertikalno naviše silom pritiska), aksijalno zatezanje (aksijalna nosivost ATT utvrđuje se opterećenjem šipa vertikalno naviše silom zatezanja) i horizontalno dejstvo savijanjem (horizontalna nosivost LLT utvrđuje se opterećenjem šipa u horizontalnom pravcu silom pritiska). U zavisnosti od formulisanog programa ispitivanja šipa, funkcija šipa može biti: probni ili testni šip (šip se izlaže opterećenju do dostizanja graničnog stanja nosivosti - destruktivni test nosivosti (DT) ili eventualno nedestruktivni test nosivosti (NDT)) i radni ili eksploatacioni šip (šip se izlaže opterećenju do dostizanja projektne nosivosti, a čija je uloga da dokaže nivo projektne nosivosti šipa i ponašanje šipa pri nivou projektne nosivosti - nedestruktivni test nosivosti (NDT)). Spoljašnje dejstvo - opterećenje, gotovo kod svih šipova, aplicira se na glavu šipa, a sam karakter dejstva može biti sledeći: statičko (SLT, BDSLT, ATT, LLT), dinamičko (DLT, PDT, HLT) i prelazna kategorija statičko-dinamičko (SNLT). Spoljašnje dejstvo može biti

3 PILE LOAD TESTING

Since a larger number of pile load tests has been developed, the conducted classification is presented as the following:

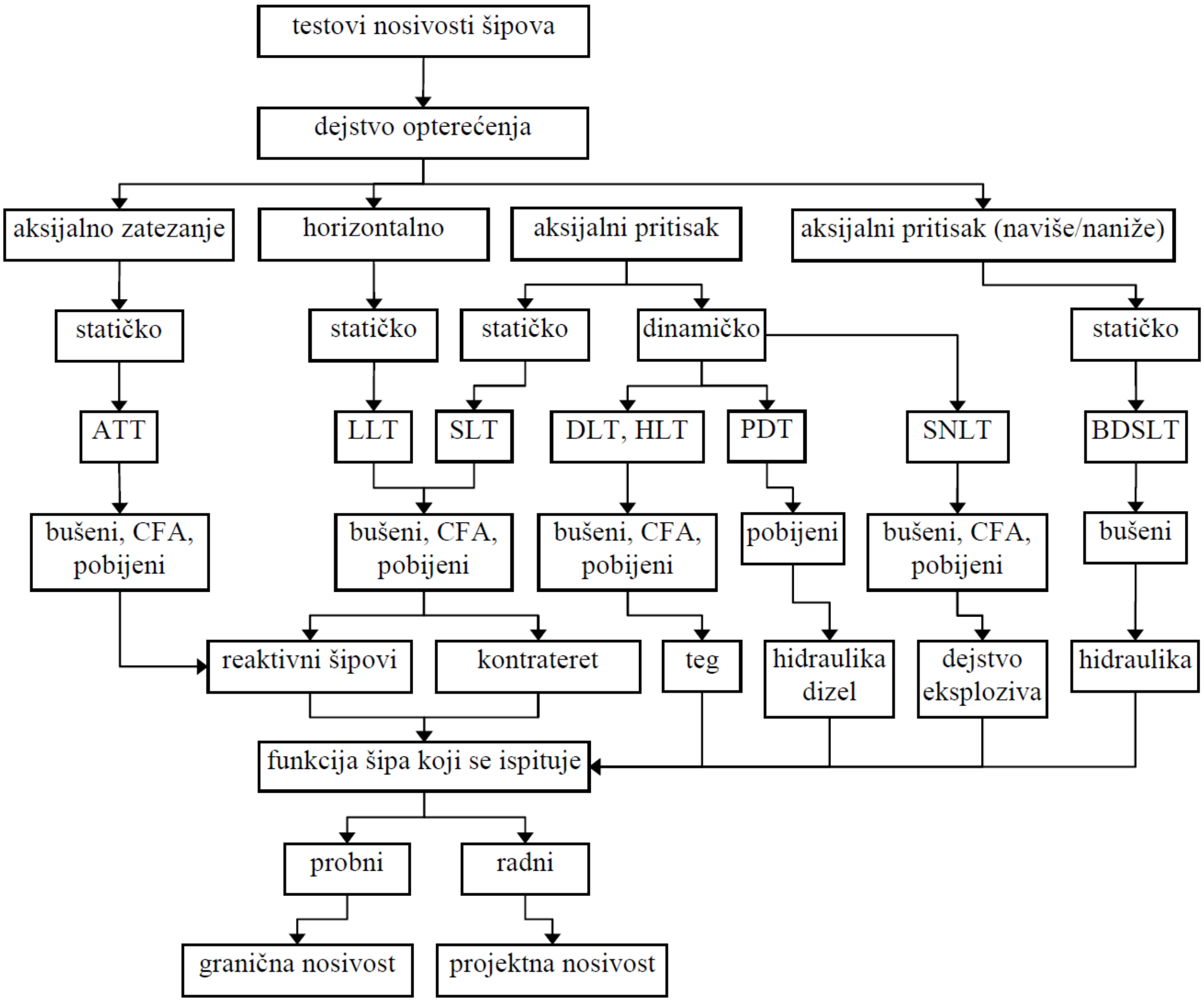
- SLT - *Static Load Test*,
- DLT - *Dynamic Load Test*,
- PDT - *Pile Driving Test*,
- HLT - *Hybridnamic Load Test*,
- SNLT - *Statnamic Load Test*,
- BDSLT - *Bi-Directional Static Load Test*,
- ATT - *Axial Tension Test*,
- LLT - *Lateral Load Test*.

Capacity types examined through these tests are classified into three groups: axial pressure (in case of all the tests of this type SLT, DLT, PDT, HLT, SNLT the axial load carrying capacity is determined by loading the pile vertically downwards by the force of pressure, while in the BDSLT case the pile is loaded vertically downwards and vertically upwards by the force of pressure), axial tension (axial load carrying capacity ATT is determined by loading the pile vertically upwards by the tension force), and the lateral bending effect (lateral load capacity LLT is determined by loading the pile in the horizontal direction by force of pressure). Depending on the formulated pile testing programme, the function of the pile can be: experimental or test pile (the pile is being loaded until the load capacity limit is reached by destructive testing (DT) or possibly non-destructive testing (NDT)), and working or exploitation pile (the pile is being loaded until the design load is reached, which has the role to prove the level of pile design load and the pile behaviour under the design load by non-destructive testing (NDT)). The external action - the load, in the case of almost all piles, is applied onto the head of the pile, and the type of the action can be: static (SLT, BDSLT, ATT, LLT), dynamic (DLT, PDT, HLT) and a transition static-dynamic category (SNLT). The external effect can proizvod: sopstvene težine kontratereta kod testova aksijalne nosivosti - sila pritiska šipova (SLT), reakcije od reaktivnih šipova kod testova aksijalne nosivosti - sila pritiska ili zatezanja šipova (SLT, ATT), reakcije pritiska u (približnom) nivou baze šipa (BDSLT), reakcije od kontratereta kod horizontalnog dejstva (LLT), reakcije od reaktivnih šipova kod horizontalnog dejstva (LLT), sile udara tega koji pada sa određene visine vertikalno naniže (DLT, PDT, HLT), sile od dejstva eksplozije i odbijanja tega vertikalno naviše (SNLT) i kontinualnih vibracija (PDT). Na slici 4 prikazan je dijagram toka opšte klasifikacije tipova testova nosivosti šipova.

Statički test opterećenja šipa (SLT) pripada grupi najpouzdanijih visokodilatacionih testova (HST) za utvrđivanje nosivosti šipova, ali, u pogledu pripreme i toka ispitivanja, ovo je najzahtevniji test [2]. Generalno razmatrajući, postoje dve varijante prema kojima se ovaj test može izvoditi: test s kontrateretom i test s reaktivnim šipovima. U prvom slučaju, pre sprovođenja testa, potrebno je dopremiti i geometrijski pravilno složiti kontrateret koji, u zavisnosti od nosivosti šipa, može biti

be a product of: the counter-load's own weight in the case of axial load tests - the pile pressure force (SLT), the reaction of reaction piles' in the case of axial load tests - the pile pressure or tension force (SLT, ATT), pressure reaction in the pile base (approximate) level (BDSLT), the counter-load's reaction in the case of lateral action (LLT), the reaction of piles in the case of lateral action (LLT), impact force of the weight falling from the certain height vertically downwards (DLT, PDT, HLT), the explosion force and the weight rebounding vertically upwards (SNLT), and continuous vibrations (PDT). Figure 4 shows a flowchart of the general pile load test types classification.

Static Load Test (SLT) belongs to the group of the most reliable high strain tests (HST) for determining pile load capacity, but, in terms of preparation and process, this is also the most demanding test [2]. Generally speaking, there are two variant ways in which this test can be performed: the test with a counter-load and the test with reaction piles. In the first case, prior to conducting the test, the counter-load, which can weigh



Slika 4. Dijagram toka opšte klasifikacije tipova testova nosivosti šipova

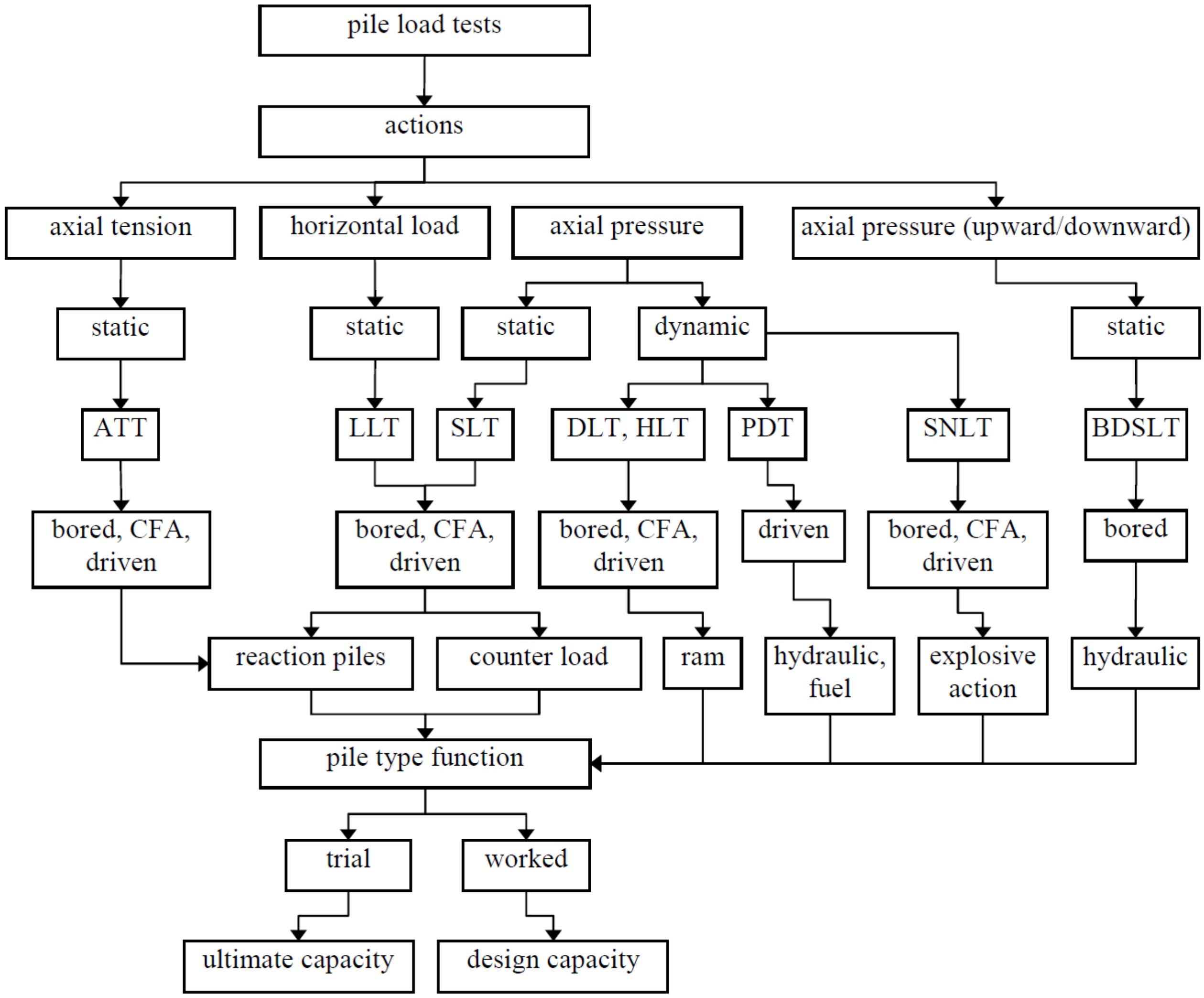


Figure 4. Flowchart of the general pile load test types classification

biti težine od nekoliko stotina do nekoliko hiljada tona. U drugom slučaju, koriste se reaktivni šipovi, koji su u toku ispitivanja opterećeni na zatezanje. Na glavu šipa postavlja se hidraulična presa preko koje se, pod inkrementalnim priraštajem pritiska ulja, istiskuje klip. Usled istiskivanja klipa i suprostavljanja težine kontratereta ili sila reaktivnih šipova, ispitivani šip se utiskuje u tlo. Primenom komparatera, prati se sleganje glave šipa. Takođe, primenom geodetskih uređaja, prati se sleganje šipa preko mernih letvi, tako da se komparacijom i naknadnom korekcijom merenja komparaterima utvrđuju krajnje vrednosti rezultata sleganja. Na osnovu sprovedenog testa, uspostavlja se relacija opterećenje-sleganje putem krive probnog opterećenja, a zatim se određuje nosivost šipa nekom od matematičkih metoda. Statički test opterećenja šipa (SLT) može se sprovoditi primenom dva varijantna rešenja: test s kontrateretom (dejstvo pritiska se realizuje usled odupiranja prese o dejstvo sopstvene težine kontratereta) i test s reaktivnim šipovima (dejstvo pritiska se realizuje usled odupiranja prese o poprečnu čeličnu gredu/traverzu koja je povezana s reaktivnim šipovima). Pod terminom ispitivanje nosivosti šipa,

between a few hundred and a few thousand tons, depending on the pile load capacity, needs to be delivered and arranged into regular geometric shapes. In the second case, we use reaction piles, which sustain tension force through the course of testing. A hydraulic press is placed on the pile head, and the piston is pressed out through it, under the incremental increase in the pressure of the oil. As a consequence of piston ejection and the opposing weight of the counter-load or the reaction piles' forces, the pile is pressed into the ground. Pile head settlement is tracked via comparators. Also, through the use of geodetic instruments we can track the pile settlement by means of measuring battens, so that by comparing and subsequently correcting the measurements, the comparators can determine the final values of settlement results. On the basis of the conducted test, load-to-settlement relation is established by means of trial-load curve, and then the pile load capacity is determined through one of the mathematical methods.

SLT can be conducted through implementation of two variant solutions: the test with a counter-load (the pressure effect is realised as a consequence of the podrazumeva se utvrđivanje intenziteta reaktivnih sila šipa u kumulativnoj formi (po omotaču i bazi). Statičkim testom opterećenja šipa (SLT) utvrđuje se aksijalna vertikalna nosivost šipa (bazom i omotačem) na statičku silu pritiska (vertikalno naniže), a koja se aplicira na glavu šipa, pri čemu postoje dve opcije: da se šip ispituje na opterećenje pritiskom do dostizanja granične nosivosti (probni šip), a koja je prethodno određena u funkciji faktorisane vrednosti zahtevane projektne nosivosti (faktor sigurnosti jeste od 2 do 3) ili da se šip ispituje na opterećenje pritiskom do dostizanja projektne nosivosti (radni šip), a koja je prethodno određena u funkciji faktorisane vrednosti zahtevane projektne nosivosti i čija je uloga samo da dokaže nivo projektne nosivosti šipa i ponašanje šipa pri nivou projektne nosivosti (faktor sigurnosti minimalno 1.1).

Statički test opterećenja šipa (SLT) zasniva se na: nelinearnoj teoriji, interakciji konstrukcija-tlo i teoriji i obradi signala. Prema nelinearnoj teoriji, uzima se u obzir to što je konstitutivni model ponašanja šipa i tla nelinearno-plastičan. Prema interakciji konstrukcija-tlo, razmatraju se spregnut problem statičke interakcije i reakcije dva medijuma (šip i tlo), bitno različitih fizičko-mehaničkih karakteristika. Prema teoriji i obradi signala, razmatraju se digitalizacija i procesiranje signala, s ciljem dobijanja odgovarajućih konačnih rezultata primenljivih u građevinskoj inženjerskoj praksi, pomoću kojih se donose odluke o nosivosti šipa. Potiskivanjem klipa iz cilindra prese, na glavu šipa, aplicira se sila čiji se intenzitet inkrementalno povećava i smanjuje putem: ciklusa samo jednog opterećenja i jednog rasterećenja ili većeg broja ciklusa opterećenja i rasterećenja, koji mogu biti različitih maksimalnih intenziteta. Za probni šip opterećenje aplicira se inkrementalno do maksimalne sile definisane programom ispitivanja, a koja treba da je jednaka 200% vrednosti projektne nosivosti šipa. Za radni šip opterećenje se aplicira inkrementalno do maksimalne sile definisane programom ispitivanja, koja treba da je jednaka faktorisanoj vrednosti projektne nosivosti šipa. Vrednost inkrementa opterećenja treba da je jednaka 25% vrednosti ukupnog opterećenja. Nakon potpunog apliciranja opterećenja, sprovodi se potpuno raterećenje. Naredni ciklus opterećenja može se, takođe, sprovoditi do maksimalne sile definisane programom ispitivanja. Vrednost svakog inkrementa aplicirane sile treba održavati na konstantnoj vrednosti, uz postizanje uslova da sleganje bude manje od 0.25mm za vreme od 1 sata, ali ne duže od 2 sata. Vrednost aplicirane sile koja odgovara projektnoj nosivosti šipa i maksimalnoj sili definisanoj programom ispitivanja treba održavati na konstantnoj vrednosti, uz postizanje uslova da sleganje bude manje od 0.25mm za vreme od 2 sata, ali ne duže od 4 sata. Nakon sprovedenog statičkog testa opterećenja šipa (SLT), obrađuju se podaci merenja i konstruiše finalna kriva opterećenje-sleganje, a zatim se određuje granična nosivost šipa.

press resistance to the counter-load's own weight) and the test with reaction piles (the pressure effect is realised as a consequence of the press resistance to the steel crossbeam/transverse beam which is connected to the reaction piles). The term pile load testing stands for determining the intensity of pile reaction forces in a cumulative form (via the shaft and the base). SLT determines the axial vertical load carrying capacity (by the base and the shaft) on the static pressure force (vertically downwards), and which is applied to the pile head, with two options: to test the pile by applying pressure until the limit carrying capacity is reached (test pile), and which has been previously established in the factorised value function of the required design load (the safety factor is between 2 and 3), or to test the pile by applying pressure until the design load is reached (working pile), which has been previously established in the factorised value function of the required design load and with the sole role of demonstrating the level of the pile design load and the pile behaviour while under the design load (safety factor minimum 1.1). SLT is based on: non-linear theory, soil-structure interaction, and theory and signal processing. According to the non-linear theory, we take into account the fact that the constitutive models of the pile and soil are non-linear-plastic. According to soil-structure interaction, the coupled problem of static interaction and reaction of two mediums (pile and soil), with significantly different physical-mechanical properties are considered. According to the theory and signal processing, signal digitalisation and processing with the aim of acquiring corresponding final results are considered. They are practically applicable in civil engineering, and with their help the decisions regarding the pile load capacity conditions are reached. A force is applied through pushing the piston from the press cylinder, to the pile head, whose intensity incrementally increases and decreases through the cycle of a single loading and a single unloading or a higher number of loading and unloading cycles, which can have different maximum intensities. In the case of a trial pile, the load is applied incrementally to the maximum force defined in the research programme, and which should be equal to 200% of the pile design load value. In the case of a working pile, the load is applied incrementally to the maximum force defined in the research programme, and which should be equal to the factorised value of the pile design load. The load increment value should be equal to 25% of the total load value. After a complete load application, complete unloading is carried out. The next loading cycle can also be carried out to the maximum force defined by the research program. The value of each increment of the applied force should be kept at a constant value, while achieving the conditions for subsidence to be less than 0.25mm during a 1 hour period, but no longer than 2 hours. The applied force value corresponding to the pile design load and the maximum force defined in the research program should be kept at a constant value, while achieving the conditions for subsidence to be less than 0.25mm during a 2 hour period, but no longer than 4 hours. After conducting the SLT the measurement data are processed and the final load-settlement curve is constructed, and then the ultimate load of the pile is determined.

Dinamički test opterećenja šipa (DLT) pripada grupi visokodilatacionih testova (HST) za utvrđivanje nosivosti šipova, pa je, s obzirom na vreme pripreme i toka ispitivanja, ovo dosta brži test u poređenju sa SLT [8]. Postoji nekoliko varijanti ovog testa, ali se ovde razmatraju: test opterećenja šipa pri udaru tega, test opterećenja šipa pri pobijanju i kombinovani testovi opterećenja šipa sa dinamičkim dejstvom. Dinamički test opterećenja šipa (DLT) zasniva se na utvrđivanju statičke nosivosti šipa pri dinamičkom dejstvu. Generalno razmatrajući, postoje dve varijante prema kojima se može ovaj test sprovoditi: test sa sopstvenim sistemom za podizanje tega i test sa pomoćnim sistemom za podizanje tega na određenu visinu. Zajedničko za obe ove varijante jeste to što se teg izlaže slobodnom padu sa određene visine i tako, usled udara tega o glavu šipa, izazove dinamičko dejstvo u šipu. U prvom slučaju, u okviru opreme za dinamičko ispitivanje, postoji sistem kojim se teg podiže na određenu visinu i tu zaustavlja kočnicama s varijantama: sa opremom samo povezanom za glavu šipa i opremom koja se oslanja na okolno tlo (za veće težine tega). U drugom slučaju, koriste se autodizalica, kran ili neko drugo slično vozilo kojim se teg podiže na određenu visinu, zadržava, te tako dozvoljava se njegovo slobodno padanje.

Dinamički test opterećenja šipa pri pobijanju (PDT) zasniva se na analizi nosivosti i monitoringu ponašanja šipa prilikom pobijanja. U odnosu na dinamički test opterećenja šipa (DLT), gde se koristi posebna oprema za ispitivanje nosivosti šipa, ovaj test se zasniva na korišćenju mašina za pobijanje šipova kao spoljašnjeg dinamičkog dejstva. Generalno razmatrajući, postoje dve varijante prema kojima se može ovaj test izvoditi: test s mašinom za pobijanje šipa pod udarcima tega (cikličan proces) i test s mašinom za pobijanje šipa vibracijama (kontinualan proces). Prednost dinamičkog testa pri pobijanju šipa (PDT), u odnosu na dinamički test šipa (DLT), jeste to što se kontinualno, u fazama pobijanja šipa kroz tlo, može pratiti i određivati sila reakcije u bazi i po omotaču. Takođe, mogu se pratiti i drugi efekti, kao što su provera integriteta šipa (npr. velike prsline, oštećenja, lom šipa).

Hibridnamički test opterećenja šipa (HLT) naziva se i brzi test opterećenja šipa (RLT), a u osnovi koristi princip dinamičkog testa opterećenja šipa (DLT), s tim što su eliminisani efekti naknadnih udara spoljašnjeg dejstva. Na taj način, naponsko stanje postaje sličnije naponskom stanju pri statičkom testu opterećenja šipa (SLT). U slučaju ovog testa postoje tri opcije za eliminisanje naknadnih udara: test s hibridnamičkim jastukom, test s hidrauličnim kočionim mehanizmom i test s kombinovanim sistemom (hibridnamički jastuk i hidraulični kočioni mehanizam). Zahvaljujući razvijenom hibridnamičkom jastuku, vreme apliciranja kinetičke energije, od spoljašnjeg dejstva (tega), na glavu šipa je znatnije produženo, nego što je to slučaj kod dinamičkog testa opterećenja šipa (DLT). Hibridnamički jastuk je oblika saća i sastoji se iz čeličnih ploča, ćelija ispunjenih vazduhom i elastomera koji ima svojstvo gume. Ovako konstruisan hibridnamički jastuk sprečava dodatno stvaranje naknadnih udara pri odbijanju tega od glave šipa, budući da ovo stvara dodatan negativan efekat u toku ispitivanja. U drugom slučaju ovaj test se sprovodi

Dynamic Load Test (DLT) belongs to the group of high strain tests (HST) for determining the pile load capacity, so, considering the amount of time that the preparation and the process take, this is a much faster test compared to the SLT [8]. There are several variants of this test, but the ones considered here are: pile load test in the case of weight (ram) impact, pile load test in the case of pile driving, and combined pile load tests with a dynamic effect. DLT is based on determining the static pile load capacity in the case of a dynamic effect. Generally speaking, there are two variant ways in which this test can be performed: a test with its own system for lifting the weight (ram), and a test with an auxiliary system for lifting the weight (ram) to a certain height. What these two variants have in common is that the weight is exposed to a free fall from a certain height, and thus, due to the impact on the pile head, it causes a dynamic effect in the pile. In the first case, as a part of the dynamic testing equipment, there is a system by which the weight is lifted to a certain height and then stopped there by brakes with variants: with the equipment only connected to the pile head, and the equipment that leans on the surrounding ground (for larger weights). In the second case, we use a crane, auto crane, or a similar vehicle, which helps lift the weight to a certain height, keep it, and then allowing its free fall.

Pile Driving Test (PDT) is based on load capacity analysis and pile behaviour monitoring while it is driven into the ground. Compared to the DLT, where special equipment is used for pile load capacity testing, this test is based on the use of driving machines as an external dynamic effect. Generally speaking, there are two variant ways in which this test can be performed: a test with a machine for pile driving under a weight impacts (cyclic process), and a test with a machine for pile driving through vibrations (continuous process). The advantage of the PDT over the DLT is that, in the stages of driving the pile into the soil, it is possible to continuously monitor and determine the reaction force in the base and the shaft. Also, other effects can be monitored, such as pile integrity checks (large cracks, damage, breakage).

Hybridnamic Load Test (HLT) is also called the Rapid Load Test (RLT), and basically uses the DLT principle, but eliminates the additional effects of an external effect impacts. Thus, the stress-state becomes more similar to the stress-state during to the SLT. In the case of this test, there are three options for eliminating the additional impacts: the hybridinamic cushion test, the hydraulic brake mechanism test, and a combined system test (hybridinamic cushion and hydraulic brake mechanism). Thanks to the developed hybridinamic cushion, the kinetic energy application time, from the external effect (the weight), onto the pile head is considerably extended compared to the DLT. The hybridinamic cushion has a honeycomb shape and consists of steel plates, air-filled cells and elastomer with rubber properties. A hybridinamic cushion structured in this way prevents additional impacts as the weight rebounds after hitting the pile head, since this would create an additional negative effect during the testing. In the second case, this test is conducted by allowing the external action (weight) impact and its rebounding after hitting the pile head, and then stopping it by means of the hydraulic braking mechanism, so as to prevent the tako što se nakon udara spoljašnjeg dejstva (tega) i njegovog odbijanja od glave šipa, on zadržava hidrauličnim kočionim mehanizmom, tako da se ne dozvoljavaju naknadni udari pri odbijanju tega od glave šipa.

Statnamički test opterećenja šipa (SNLT) u osnovi koristi princip dinamičkog testa opterećenja šipa (DLT), s tim što su eliminisani efekti naknadnih udara spoljašnjeg dejstva, međutim, ovo dejstvo kojim se deluje na glavu šipa realizuje se eksplozivnim dejstvom goriva [1]. Generalno razmatrajući, postoje dve varijante prema kojima se ovaj test može izvoditi: test kod koga se teg eksplozivnim dejstvom podiže vertikalno naviše, a zatim slobodno pada na šljunak (ili sličan materijal), koji se nalazi između glave šipa i tega, kao i test kod koga se teg eksplozivnim dejstvom podiže vertikalno naviše, a zatim hidrauličnim kočionim mehanizmom zaustavlja na određenoj visini, ne dozvoljavajući mu da slobodno padne. Primenom šljunka ili hidrauličnim kočionim mehanizmom, ublažava se efekat dinamičkog dejstva naknadnih udaraca koji bi se ralizovali odbijanjem tega od glave šipa, dok se šip pod dejstvom sile potiska pomera vertikalno naniže. Reaktivna sila koja se aplicira na glavu šipa, usled eksplozivnog dejstva pri podizanju tega vertikalno naviše, veća je nekoliko desetina do stotina puta od sopstvene težine tega. Dinamički testovi opterećenja šipa zasnivaju se na: dinamici kretanja krutog tela, talasnoj teoriji, metodi karakteristika, nelinearnoj teoriji, dinamici konstrukcija, interakciji konstrukcija-tlo i teoriji i obradi signala. Prema dinamici kretanja krutog tela, razmatra se apliciranje spoljašnjeg dejstva udarom idealno krutog tela (tega) o glavu šipa. Prema talasnoj teoriji, razmatraju se aspekti propagacije talasa kroz šip i tlo. Prema metodi karakteristika, razmatraju se aspekti kretanja odlazećih i dolazećih talasa u šipu. Prema nelinearnoj teoriji, uzima se u obzir to što je konstitutivni model ponašanja šipa i tla nelinearno-plastičan. Prema dinamici konstrukcija, razmatraju se oscilacije šipa u interakciji s tlom u vremenskom domenu. Prema interakciji konstrukcija - tlo, razmatraju se spregnut problem statičke i dinamičke interakcije i reakcije dva medijuma (šip i tlo), bitno različitih fizičko-mehaničkih karakteristika. Prema teoriji i obradi signala, razmatraju se digitalizacija, procesiranje i kompatibilizacija (*signal matching*) signala, s ciljem dobijanja odgovarajućih konačnih rezultata primenljivih u građevinskoj inženjerskoj praksi, pomoću kojih se donose odluke o nosivosti šipa. Dinamički testovi opterećenja šipa sprovode se u nekoliko koraka: analiziraju se svi relevantni podaci u pasošu šipa i geotehničkom elaboratu, setuje se oprema, podaci i parametri za šip koji se ispituje, ukoliko se test sprovodi s tegom, teg se podiže na odgovarajuću visinu, a ukoliko se sprovodi pobijanjem ili vibracijama, kontinualno se prati stanje sile reakcije. Ukoliko se test sprovodi s eksplozivom, preduzimaju se sve prethodne mere bezbednosti i instalacije punjenja eksploziva, postupak se ponavlja nekoliko puta (maksimalno 10 puta ukupno i maksimalno 2 puta za jednu istu visinu). Za sve DLT testove analiziraju se: količina unete kinetičke energije, nivo napona zatezanja, nivo napona pritiska, apsolutne maksimalne vrednosti akceleracija, maksimalne vrednosti elasto-plastičnih deformacija, pojava negativnih vrednosti u dolazećem signalu sile u šipu i slično. Ako se pokaže potrebnim koriguju se parametri koji su korišćeni

additional impacts as the weight rebounds after hitting the pile head.

Statnamic Load Test (SNLT) basically uses the principle of the DLT, but it eliminates the additional effects of the external action impacts, however, this action that the head of the pile sustains is realized by the explosive action of the fuel [1]. Generally speaking, there are two variant ways in which this test can be performed: a test where a weight is lifted by an explosive action vertically upwards, and then falls freely on gravel (or a similar material) placed between the pile head and the weight, and a test where a weight is lifted by an explosive action vertically upwards, and then stopped at a certain height by a hydraulic braking mechanism, not allowing it to fall freely. The gravel or hydraulic braking mechanism mitigates the dynamic action effect of the additional impacts, which would be realised by the weight rebounding after hitting the pile head, while the pile under the thrust force moves vertically downwards. The reaction force applied to the pile head, as a consequence of the explosive action when the weight is lifted vertically upwards, is tens or hundreds of times larger than the weight's own weight. All DLT tests are based on: rigid-body dynamics, wave theory, method of characteristics, non-linear theory, dynamics of structures, soil-structure interaction and theory and signal processing. According to the rigid-body dynamics, we consider the application of an external action through an impact from an ideally rigid body (weight) onto the pile head. According to the wave theory, the aspects of wave propagation through the pile and the soil are considered. According to the method of characteristics, the propagation aspects of downward and upward waves in the pile are considered. According to the non-linear theory, we take into account the fact that the constitutive models of the pile and soil are non-linear-plastic. According to the dynamics of structures, the pile oscillations during the interaction with the soil within the time domain are considered. According to soil-structure interaction, the coupled problem of static and dynamic interaction and reactions of two mediums (pile and soil) with significantly different physical-mechanical properties are considered. According to the theory and signal processing, signal digitalisation, processing and signal matchnig are considered with the aim of acquiring corresponding final results, practically applicable in civil engineering, and with the help of which the decisions regarding the pile load capacity conditions are reached. Dynamic load tests are carried out in several steps: all the relevant data in the pile's passport and geotechnical study are analysed, the equipment, data, and parameters are set for the pile being tested, if the test is carried out with a weight, the weight is raised to the appropriate height, and if it is carried out by driving the pile or through vibrations, the state of the reaction force is continuously being monitored. If the test is carried out with an explosive, we take all preceding safety measures and explosive loading installations, the procedure is repeated several times (maximum of 10 times in total and maximum of 2 times for the same height). For all DLT tests we analyse: the amount of kinetic energy input, the tension level, the pressure level, absolute maximum acceleration values, maximum elastoplastic deformation values, the occurrence of negative values in the upward force signal in the pile, and the like. If necessary, the pri inicijalnom setovanju, radi dobijanja što tačnijih rezultata ispitivanja, konstruiše se reprezentativni dijagram sila dobijenih proračunom akceleracija i dilatacija i naknadno se sprovodi obrada podataka dobijenih ispitivanja u cilju utvrđivanja nosivosti šipa. Proračun nosivosti šipa sprovodi se primenom indirektnih metoda, koje se zasnivaju na iterativnoj kompatibilizaciji numeričkog nelinearnog histerezisnog modela interakcije šip-tlo prema *in-situ* merenom signalu dinamičkog testa, tako što se eliminiše dinamička komponenta nosivosti šipa, a zadržava statička komponenta nosivosti šipa.

Bidirekcioni statički test opterećenja šipa (BDSLT) jeste test novije generacije, koji ne zahteva angažovanje kontratereta ili reaktivnih šipova, a pripada grupi visokodilatacionih testova (HST) [9]. Specifičnost ovog testa jeste to što se glavni deo opreme za ispitivanje testa (*Osterberg*-ova ćelija) ugrađuje u telo bušenih šipova, pa se ovakvi šipovi ne mogu dalje koristiti u eksploataciji kao primarni noseći elementi dubokog fundiranja. Bidirekcioni statički test opterećenja šipa (BDSLT) zasniva se na: nelinearnoj teoriji, interakciji konstrukcija-tlo i teoriji i obradi signala. Prema nelinearnoj teoriji, uzima se u obzir to što je konstitutivni model ponašanja šipa i tla nelinearno-plastičan. Prema interakciji konstrukcija-tlo, razmatraju se spregnut problem statičke interakcije i reakcije dva medijuma (šip i tlo), bitno različitih fizičko-mehaničkih karakteristika. Prema teoriji i obradi signala, razmatraju se digitalizacija i procesiranje signala, s ciljem dobijanja odgovarajućih konačnih rezultata primenljivih u građevinskoj inženjerskoj praksi, pomoću kojih se donose odluke o nosivosti šipa. Postupak pripreme i sprovođenja testa odvija se u nekoliko koraka: za formirani armaturni koš zavaruju se dve kružne čelične ploče, između kojih se postavlja/ju *Osterberg*-ova/e ćelija/e do kojih se dovode creva za hidrauliku i ekstenziometri, ugrađuje se armaturni koš i izliva betonska mešavina, nakon očvršćavanja betona pristupa se sprovođenju testa, pod dejstvom hidrauličnog pritiska dolazi do loma u betonu na mestu gde su postavljene *Osterberg*-ove ćelije, šip se potiskuje vertikalno naviše, a zatim naniže, dok se primenom instrumenata prate pritisak u *Osterberg*-ovoj ćeliji i deformacije (sleganje/izdizanje) šipa. Potiskivanjem klipa iz cilindra prese (*Osterberg*-ove ćelije), aplicira se sila po poprečnom preseku u nivou baze (nešto iznad baze), čiji se intenzitet inkrementalno povećava i smanjuje putem: ciklusa samo jednog opterećenja i jednog rasterećenja ili većeg broja ciklusa opterećenja i rasterećenja, koji mogu biti različitih maksimalnih intenziteta.

Test aksijalnog zatezanja šipa (ATT) pripada grupi visokodilatacionih testova (HST) za utvrđivanje nosivosti šipova, koji se sprovodi primenom reaktivnih šipova izloženih dejstvu aksijalnog pritiska, dok je šip koji se ispituje, izložen dejstvu aksijalne sile zatezanja [3]. Test aksijalnog zatezanja šipa (ATT) može se sprovoditi primenom dva varijantna rešenja: test s presom postavljenom na glavnu čeličnu gredu/traverzu iznad šipa koji se ispituje (dejstvom sile pritiska, vertikalno naviše, podižu se poprečne čelične gredice, pri čemu se ispitni šip izlaže sili zatezanja preko ankera ugrađenih u njemu, a povezanih s poprečnim čeličnim gredicama),

parameters used in the initial setup are corrected, in order to obtain the most accurate results of the test, a representative diagram of the forces obtained through the acceleration and strain calculations are constructed, and the data acquired through testing are additionally processed with the aim of determining the pile load capacity. The pile load capacity calculation is conducted by using indirect methods, which are based on the iterative compatibilisation of the numerical non-linear hysteresis model of the soil-structure interaction according to the in-situ measured signal of the dynamic test, by eliminating the dynamic component of the pile load capacity and retaining the static component of the pile load capacity.

Bi-Directional Static Load Test (BDSLT) is a new generation test that does not require the use of counterload or reaction piles, and belongs to a group of high strain test (HST) [9]. What's specific about this test is that the main piece of the test examination equipment (*Osterberg* cell) is incorporated into the body of bored piles, so these piles can no longer be used in exploitation as the primary deep foundation supporting elements. BDSLT is based on: non-linear theory, soil-structure interaction, and theory and signal processing. According to the non-linear theory, we take into account the fact that the constitutive models of the pile and soil are non-linear-plastic. According to soil-structure interaction, account the fact that the constitutive models of the pile and soil are non-linear-plastic. According to soil-structure interaction, the coupled problem of static interaction and reactions of two mediums (pile and soil) with significantly different physical-mechanical properties are considered. According to the theory and signal processing, signal digitalisation and processing with the aim of acquiring corresponding final results are considered. They are practically applicable in civil engineering, and with their help the decisions regarding the pile load capacity conditions are reached. The process of preparing and conducting the test is done in several steps. Two circular steel plates are welded onto the formed reinforcement cage, while also placing *Osterberg* cell(s), which are connected to hydraulic hoses and extensometers. Between the two plates, the reinforcement cage is installed and the concrete mix poured in. After concrete curing the test is conducted. The concrete breaks at the spots where the *Osterberg* cells have been placed under hydraulic pressure. The pile is pushed vertically upwards and then downwards, while the pressure in the *Osterberg* cell and pile deformations (subsidence/lifting) are monitored through the instruments. By pressing the piston from the press cylinder (*Osterberg* cell), force is applied on the cross-section at the base level (somewhat above the base), and the intensity of which incrementally increases and decreases through: a single loading and single unloading cycle or a larger number of loading and unloading cycles, which can have different maximum intensities.

Axial Tension Test (ATT) belongs to the group of high strain tests (HST) for determining the pile load capacity, which is carried out through the use of reaction piles exposed to axial pressure, while the pile being examined is exposed to the axial tension force [3]. ATT can be carried out through the use of two variant solutions: a test with a press placed on the main steel beam/transverse beam above the pile being tested (the kao i test s presama postavljenim na glave reaktivnih šipova ispod glavne čelične grede/traverze (dejstvom sile pritiska, vertikalno naviše, podiže se glavna čelična greda/traverza, pri čemu se ispitni šip izlaže sili zatezanja preko ankera ugrađenih u njemu, a povezanih s poprečnim čeličnim gredicama koje su oslonjene na glavnu čeličnu gredu/traverzu). Potiskivanjem klipa iz cilindra prese, podižu se anker nosači i aksijalno zateže šip koji se ispituje, pri čemu se intenzitet sile inkrementalno povećava i smanjuje putem: ciklusa samo jednog opterećenja i jednog rasterećenja ili većeg broja ciklusa opterećenja i rasterećenja, koji mogu biti različitih maksimalnih intenziteta. Test aksijalnog zatezanja šipa (ATT) zasniva se na: nelinearnoj teoriji, interakciji konstrukcija-tlo i teoriji i obradi signala. Prema nelinearnoj teoriji, uzima se u obzir to što je konstitutivni model ponašanja šipa i tla nelinearno-plastičan. Prema interakciji konstrukcija-tlo, razmatraju se spregnut problem statičke interakcije i reakcije dva medijuma (šip i tlo), bitno različitih fizičko-mehaničkih karakteristika. Prema teoriji i obradi signala, razmatraju se digitalizacija i procesiranje signala, s ciljem dobijanja odgovarajućih konačnih rezultata primenljivih u građevinskoj inženjerskoj praksi, pomoću kojih se donose odluke o nosivosti šipa.

Test horizontalnog opterećenja šipa (LLT) pripada grupi visokodilatacionih testova (HST) za utvrđivanje nosivosti šipova, koji se, u najvećem broju slučajeva, sprovodi primenom kontratereta koji predstavlja oslonac za horizontalno dejstvo na šip koji se ispituje ili primenom reaktivnih šipova, koji preuzimaju ulogu oslonca za horizontalno dejstvo [4]. Ovim testom, između ostalog, mogu se utvrditi vrednosti: koeficijenta horizontalne rekacije tla, napona pritiska i zatezanja (od savijanja) u šipu i horizontalnog pomeranja glave šipa. Test horizontalnog opterećenja šipa (LLT) može se sprovoditi primenom dva varijantna rešenja: test s kontrateretom (dejstvo horizontalnog opterećenja realizuje se usled horizontalnog odupiranja prese o dejstvo sopstvene težine kontratereta) i test s reaktivnim šipovima (dejstvo horizontalnog opterećenja realizuje se usled horizontalnog odupiranja prese o reaktivne šipove). Test horizontalnog opterećenja šipa (LLT) zasniva se na: nelinearnoj teoriji, interakciji konstrukcija-tlo i teoriji i obradi signala. Prema nelinearnoj teoriji, uzima se u obzir to što je konstitutivni model ponašanja šipa i tla nelinearno-plastičan. Prema interakciji konstrukcija-tlo, razmatraju se spregnut problem statičke interakcije i reakcije dva medijuma (šip i tlo), bitno različitih fizičko-mehaničkih karakteristika. Prema teoriji i obradi signala, razmatraju se digitalizacija i procesiranje signala, s ciljem dobijanja odgovarajućih konačnih rezultata primenljivih u građevinskoj inženjerskoj praksi, pomoću kojih se donose odluke o nosivosti šipa. Potiskivanjem klipa iz cilindra prese, na bočnu stranu glave šipa, aplicira se sila čiji se intenzitet inkrementalno povećava i smanjuje putem: ciklusa samo jednog opterećenja i jednog rasterećenja ili većeg broja ciklusa opterećenja i rasterećenja, koji mogu biti različitih maksimalnih intenziteta. Za probni šip opterećenje se aplicira inkrementalno do maksimalne sile definisane programom ispitivanja, koja treba da je jednaka 200% vrednosti projektne horizontalne nosivosti šipa. Za radni

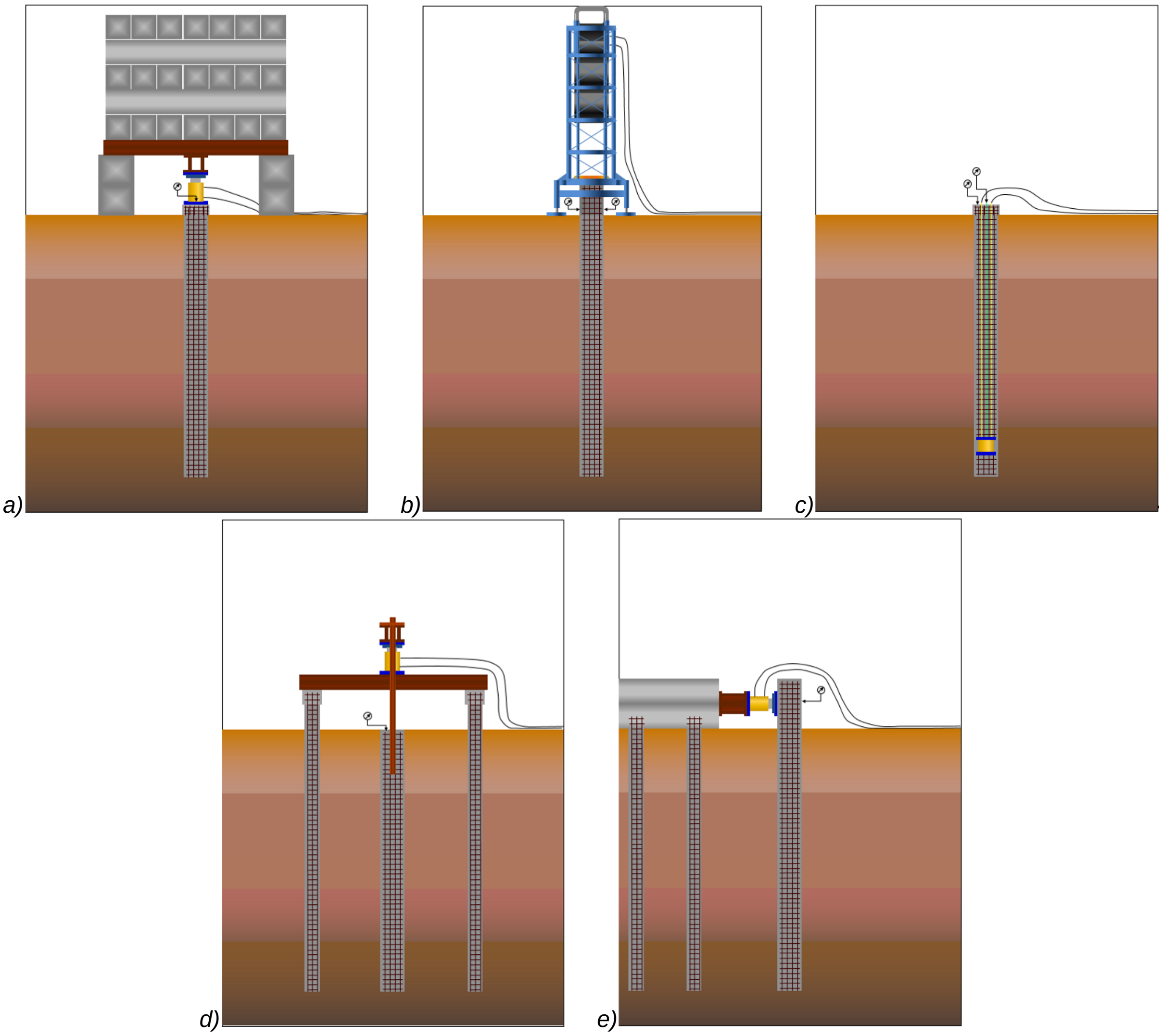
force of pressure, vertically upwards, lifts the steel cross bars, while the test pile is exposed to the tension force via the built-in anchors, which are connected to the steel cross bars), and a test with presses placed on the reaction piles' heads below the main steel beam/transverse beam (the force of the pressure, vertically upwards, lifts the main steel beam/transverse beam, while the test pile is exposed to the tension force via the built-in anchors, which are connected to the steel cross bars that are supported by the main steel beam/transverse beam). Pressing the piston from the press cylinder lifts the pile anchor beams and puts the pile being examined under the axial tension, with the force intensity incrementally increasing and decreasing through: a single loading and single unloading cycle or a larger number of loading and unloading cycles, which can have different maximum intensities. ATT is based on: non-linear theory, soil-structure interaction, and theory and signal processing. According to the non-linear theory, we take into account the fact that the constitutive models of the pile and soil are non-linear-plastic. According to soil-structure interaction, we consider the coupled problem of static interaction and reactions of two mediums (pile and soil), with significantly different physical-mechanical properties. According to the theory and signal processing, we consider signal digitalisation and processing with the aim of acquiring corresponding final results, practically applicable in civil engineering, and with the help of which the decisions regarding the pile load capacity conditions are reached.

Lateral Load Test (LLT) belongs to the group of high strain tests (HST) for determining the pile load capacity, which, in most cases, is carried out through the use of a counter-load, which is a support for the horizontal action that the tested pile sustains, or through the use of reaction piles, which take over the role of support for the horizontal action [4]. This test can determine, among other things, the values of: coefficient of horizontal subgrade reaction, stress pressure and stress tension in the pile and pile head horizontal displacement. LLT can be carried out using two variant solutions: the test with a counter-load (the lateral load effect is realised as a consequence of the lateral press resistance to the counter-load's own weight) and the test with reaction piles (the lateral load effect is realised as a consequence of the lateral press resistance to the reaction piles). LLT is based on: non-linear theory, soil-structure interaction, and theory and signal processing. According to the non-linear theory, we take into account the fact that the constitutive models of the pile and soil are non-linear-plastic. According to soil-structure interaction, we consider the coupled problem of static interaction and reactions of two mediums (pile and soil), with significantly different physical-mechanical properties. According to the theory and signal processing, we consider signal digitalisation and processing with the aim of acquiring corresponding final results, practically applicable in civil engineering, and with the help of which the decisions regarding the pile load capacity conditions are reached. By pushing the piston from the press cylinder, to the pile head lateral side, with the force intensity incrementally increasing and decreasing through: a single loading and single unloading cycle or a larger number of loading and unloading cycles, which can have different maximum intensities. In the case of šip opterećenje se aplicira inkrementalno do maksimalne sile definisane programom ispitivanja, koja treba da je jednaka faktorisanoj vrednosti projektne horizontalne nosivosti šipa. Ukoliko se razmatra konstruktivna granična nosivost, tada se ona određuje iz uslova granične nosivosti šipa na uticaje momenta savijanja. Ukoliko se razmatra geotehnička granična nosivost, tada se ona određuje iz uslova granične horizontalne otpornosti tla. Vrednost inkrementa opterećenja treba da je jednaka 25% vrednosti ukupnog opterećenja.

Na slici 5 dati su opšti šematski prikazi: a) statičkog testa opterećenja šipa (SLT) s kontrateretom, b) dinamičkog testa opterećenja šipa (DLT), c) bidirekcionog statičkog testa opterećenja šipa (BDSLT), d) testa aksijalnog zatezanja šipa (ATT) s presom postavljenom na glavnu čeličnu gredu/traverzu iznad šipa koji se ispituje, e) testa horizontalnog opterećenja šipa (LLT) s reaktivnim šipovima.

trial piles, the load is applied incrementally to the maximum force defined by the test programme, which should be equal to 200% of the pile lateral design load value. In the case of working piles, the load is applied incrementally to the maximum force defined by the test program, which should be equal to the factorised value of the pile lateral design load. If the constructive ultimate bearing capacity is considered, then it is determined from the pile ultimate bearing capacity conditions to the bending moment influence. If the geotechnical ultimate bearing capacity is considered, then it is determined from the conditions of the ultimate lateral soil resistance. The load increment value should be equal to 25% of the total load value.

Figure 5 gives a general scheme of: a) SLT with a counter-load, b) DLT, c) BDSLT, d) ATT with a press set onto the main steel beam/transverse beam above the pile being examined, e) LLT with reaction piles.



Slika 5. Opšti šematski prikazi: a) statičkog testa opterećenja šipa (SLT) sa kontrateretom, b) dinamičkog testa opterećenja šipa (DLT), c) bidirekcionog statičkog testa opterećenja šipa (BDSLT), d) testa aksijalnog zatezanja šipa (ATT) sa presom postavljenom na glavnu čeličnu gredu/traverzu iznad šipa koji se ispituje, e) testa horizontalnog opterećenja šipa (LLT) sa reaktivnim šipovima

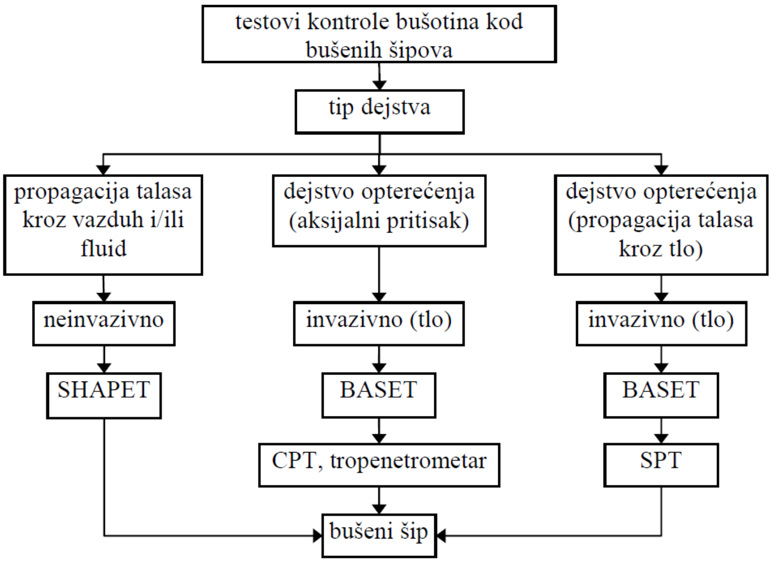
Figure 5. The general scheme of: a) SLT with a counter-load, b) DLT, c) BDSLT, d) ATT with a press set onto the main steel beam/transverse beam above the pile being examined, e) LLT with reaction piles

4 KONTROLA BUŠOTINE KOD BUŠENIH ŠIPOVA

Tipovi testova kontrole bušotina kod bušenih šipova mogu se podeliti u dve grupe: test evaluacije geometrijskih karakteristika bušotine kod bušenog šipa (SHAPET) i test evaluacije geomehaničkih karakteristika baze bušotine kod bušenog šipa (BASET). Tipovi evaluacija bušotina kod bušenih šipova, koji se utvrđuju ovim testovima, klasifikovani su u nekoliko grupa: stvarna dužina bušotine, inklinacija bušotine, promena oblika poprečnog preseka bušotine i analiza geomehaničkih karakteristika baze bušotine. Opšta metodologija ispitivanja bušotina zasniva se na: ispitivanju prilikom formiranja bušotine (neinvazivni test monitoringa bušenja) i ispitivanju nakon formirane bušotine (invazivni test tla). Na slici 6 prikazan je dijagram toka opšte klasifikacije tipova testova i kontrole bušotina kod bušenih šipova.

4 CONTROL OF BORED PILE SHAFTS

Shaft control test types in the case of bored piles can be divided into two groups: the Shaft Profile Evaluation Test (SHAPET) and the Base Evaluation Test (BASET). Shaft evaluation types in the case of bored piles, which are determined by these tests, are classified into several groups: the actual shaft length, shaft inclination, changes in the shaft cross-section shape and shaft base (geomechanical properties) analysis. The general shaft testing methodology is based on: testing during the shaft drilling (non-invasive drilling monitoring test) and testing after the drilling has been completed (invasive soil test). Figure 6 shows a flowchart of the general classification of shaft test and control types in the case of bored piles.



Slika 6. Dijagram toka opšte klasifikacije tipova testova kontrole bušotina kod bušenih šipova

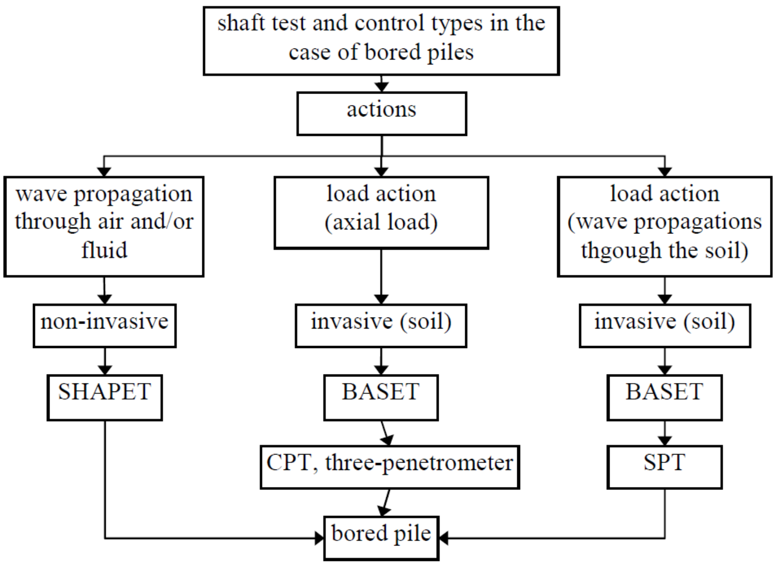


Figure 6. Flowchart of the general classification of shaft test and control types in the case of bored piles

Test evaluacije geometrijskih karakteristika bušotine kod bušenog šipa (SHAPET) zasniva se na GPS praćenju signala prilikom bušenja ili naknadnom kontrolom bušotine kod bušenih šipova, radi evaluacije devijacije vertikalnosti i dužine bušotine. Takođe, ovim testom, ultrazvučnom metodom, spuštajući sonde kontinualno naniže, utvrđuje se promena geometrijskog oblika bušotine kod bušenih šipova [30]. Ovaj test zasniva se na: talasnoj teoriji i teoriji i obradi signala. Prema talasnoj teoriji, razmatraju se aspekti propagacije talasa kroz vazduh i fluid. Prema teoriji i obradi signala, razmatraju se digitalizacija i procesiranje signala, s ciljem dobijanja odgovarajućih konačnih rezultata primenljivih u građevinskoj inženjerskoj praksi, pomoću kojih se donose odluke o evaluaciji geometrijskih karakteristika bušotine kod bušenog šipa. Test evaluacije geometrijskih karakteristika bušotine kod bušenog šipa (SHAPET) sprovodi se u nekoliko koraka: analiziraju se svi relevantni podaci u geotehničkom elaboratu, ukoliko se ispitivanje sprovodi na bušotini za bušeni šip koja tek treba da se izgradi bušenjem tla, tada se senzor za monitoring inklinacije i dubine bušotine i digitalni merač dubine bušotine pričvršćuju za nosač alata mašine kojim se buši tlo i kojim se oprema spušta do baze bušotine (naknadno, nakon izgradnje bušotine, sprovodi se 2D i/ili 3D skeniranje oblika bušotine ultrazvučnim sondama, koje se instaliraju na mašinu za bušenje tla ili drugi pomoćni sistem, kojim se oprema spušta do baze bušotine), ukoliko se ispitivanje sprovodi na formiranoj bušotini za bušeni šip, tada se senzor za monitoring inklinacije i dubine bušotine i digitalni merač dubine bušotine pričvršćuju za nosač alata mašine kojim se buši tlo ili za pomoćni sistem (tripod) kojim se oprema spušta do baze bušotine (naknadno se sprovodi 2D i/ili 3D skeniranje oblika bušotine ultrazvučnim sondama, koje se instaliraju na mašinu za bušenje tla ili drugi pomoćni sistem, kojim se oprema spušta do baze bušotine), setuju se oprema, podaci i parametri za bušotinu (bušenog šipa) koja se ispituje, konstruišu se adekvatni dijagrami kojima se prikazuju: stvarne dužine (dubine), inklinacije, devijacije vertikalnosti/horizontalnosti i promena oblika poprečnog preseka bušotine inkrementalno po dubini i naknadno se sprovodi obrada podataka dobijenih ispitivanjem.

Test evaluacije geomehaničkih karakteristika baze bušotine kod bušenog šipa (BASET) zasniva se na primeni geomehaničkih *in-situ* metoda radi evaluacije karakteristika tla u bazi bušotine. Ovaj test može se sprovoditi primenom postojećih ili unapređenih (sofisticiranijih) geomenhaničkih *in-situ* testova: test statičke penetracije (CPT), test standardne penetracije (SPT) ili test statičke penetracije sa integrisanim tropenetrometrom (simultano tropenetrometarsko ispitivanje). Test evaluacije geomehaničkih karakteristika baze bušotine kod bušenog šipa (BASET) zasniva se na: dinamici kretanja krutog tela, talasnoj teoriji, nelinearnoj teoriji, mehanici tla, dinamici tla i teoriji i obradi signala. Prema dinamici kretanja krutog tela, razmatra se apliciranje spoljašnjeg dejstva udarom idealno krutog tela (tega) o cev koja se pobija u tlo. Prema talasnoj teoriji, razmatraju se aspekti propagacije talasa kroz tlo. Prema nelinearnoj teoriji, uzima se u obzir to što je konstitutivni model ponašanja tla nelinearno-plastičan. Prema mehanici tla, razmatraju se 3D naponska stanja u tlu (u bazi bušotine). Prema dinamici tla, razmatraju se vibracije u

Shaft Profile Evaluation Test (SHAPET) is based on GPS signal tracking during drilling or the subsequent shaft control in the case of bored piles, with the aim of evaluating the deviation in verticality and the shaft length. Also, this test, through an ultrasonic method, by lowering the probes continuously downwards, determines the change in the shaft's geometric shape in the case of bored piles [30]. This test is based on: wave theory, and theory and signal processing. According to the wave theory, bored piles [30]. This test is based on: wave theory, and theory and signal processing. According to the wave theory, the aspects of wave propagation through the air and fluid are considered. According to the theory and signal processing, signal digitalisation and processing with the aim of acquiring corresponding final results are considered. They are practically applicable in civil engineering, and help in reaching decisions about the shaft profile evaluation in the case of bored piles. SHAPET is carried out in several steps: all the relevant data in the geotechnical study are analyzed, if the testing should be performed on a shaft for a bored pile which has yet to be built by drilling the soil, then the sensor for monitoring the inclination and shaft depth, and a digital shaft depth gauge are attached to the tool holder of the machine that drills the ground and lowers the equipment to the base of the shaft (subsequently, after the shaft has been constructed, we conduct 2D and/or 3D scanning of the shaft shape by ultrasonic probes, which are installed on the soil drilling machine or another auxiliary system, used for lowering the equipment to the base of the shaft), if the test is carried out on a formed drilled pile shaft, then the sensor for monitoring the inclination and shaft depth and a digital shaft depth gauge are attached to the tool holder of the machine that drills the ground or to an auxiliary system (tripod) used for lowering the equipment to the shaft base (subsequently, we conduct 2D and/or 3D scanning of the shaft shape by ultrasonic probes, which are installed on the soil drilling machine or another auxiliary system, used for lowering the equipment to the base of the shaft), the equipment, data, and parameters are set for the (bored pile) shaft being tested, adequate diagrams are constructed to show: the actual lengths (depths), inclinations, verticality/horizontality deviations, and the shaft cross-section shape changes, incrementally in depth, and we subsequently process the testing obtained data.

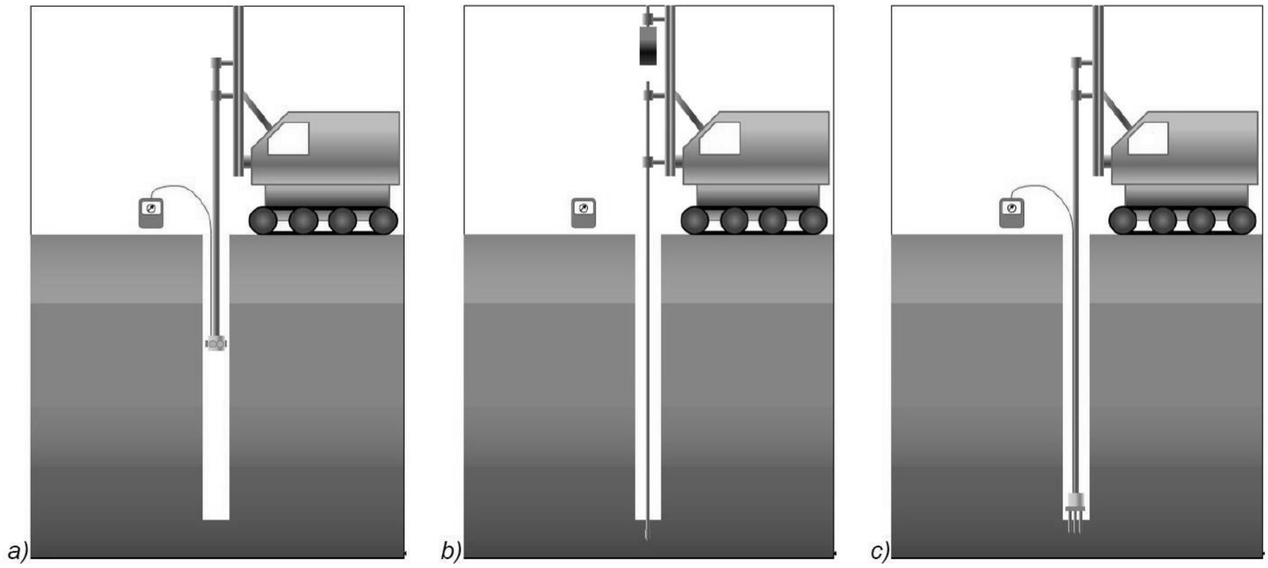
Base Evaluation Test (BASET) is based on the use of geomechanical in-situ methods with the aim of evaluating soil properties in the shaft base. This test can be carried out using the existing or improved (more sophisticated) geomechanical in-situ tests: *Cone Penetration Test* (CPT), *Standard Penetration Test* (SPT) or static penetration test with an integrated three-penetrometer (simultaneous three-penetrometric testing). BASET is based on: the rigid-body dynamics, wave theory, non-linear theory, soil mechanics, soil dynamics, and theory and signal processing. According to the rigid-body dynamics, we consider the application of an external action through an ideally rigid body (weight) impact on the tube that is being driven into the soil. According to the wave theory, we consider the aspects of wave propagation through the soil. According to the non-linear theory, we take into account the fact that the constitutive model of soil is non-linear-plastic.

tlu u vremenskom domenu i aspekti prigušenja. Prema teoriji i obradi signala, razmatraju se digitalizacija i procesiranje signala, s ciljem dobijanja odgovarajućih konačnih rezultata primenljivih u građevinskoj inženjerskoj praksi, pomoću kojih se donose odluke o evaluaciji geomehaničkih karakteristika baze bušotine kod bušenog šipa. Test statičke penetracije (CPT) i test standardne penetracije (SPT) sprovode se prema odgovarajućim standardima. Test integrisanim tropenetrometrom sprovodi se slično testu CPT, s tim što se tri penetrometra simultano utiskuju u bazu bušotine šipa.

Na slici 7 dati su opšti šematski prikazi: a) testa evaluacije geometrijskih karakteristika bušotine kod bušenog šipa (SHAPET) - evaluacija promene oblika poprečnog preseka bušotine, b) testa evaluacije geomehaničkih karakteristika baze bušotine kod bušenog šipa (BASET) - test standardne penetracije (SPT) baze, c) testa evaluacije geomehaničkih karakteristika baze bušotine kod bušenog šipa (BASET) - integrisani tropenetrometar.

According to the soil mechanics, 3D soil stress conditions are considered (in the shaft base). According to the soil dynamics, we consider soil vibrations in the time domain and the absorption aspects. According to the theory and signal processing, we consider signal digitalisation and processing with the aim of acquiring corresponding final results, practically applicable in civil engineering, and which will help in reaching decisions about the base evaluation in the case of bored piles. CPT and SPT are conducted according to corresponding standards. The integrated three-penetrometer test is performed similarly to the CPT test, while there are three penetrometers which are simultaneously pressed into the base.

Figure 7 gives a general scheme of: a) SHAPET - the evaluation of the shaft cross-section shape change, b) BASET - SPT of the base, c) BASET - integrated three-penetrometer.



Slika 7. Opšti šematski prikazi: a) testa evaluacije geometrijskih karakteristika bušotine kod bušenog šipa (SHAPET) - evaluacija promene oblika poprečnog preseka bušotine, b) testa evaluacije geomehaničkih karakteristika baze bušotine kod bušenog šipa (BASET) - test standardne penetracije (SPT) baze, c) testa evaluacije geomehaničkih karakteristika baze bušotine kod bušenog šipa (BASET) - integrisani tropenetrometar

Figure 7. The general scheme of: a) SHAPET - the evaluation of the shaft cross-section shape change, b) BASET - SPT of the base, c) BASET - integrated three-penetrometer

5 ZAKLJUČAK

Metodologija ispitivanja integriteta i nosivosti šipova definisana je procedurama propisanim u standardima o ispitivanjima šipova. Unapređivanje segmenata postojećih standarda odgleda se u sledećem: detaljnijem razjašnjenju pojedinih faza ispitivanja, redukciji i selekciji metoda i postupaka ispitivanja u okviru jednog testa, s obzirom na to što u postojećim standardima ispitivanja šipova postoji veći broj opcija ispitivanja koja se najčešće i ne koriste, prikazani su ključni elementi ispitivanja integriteta i nosivosti šipova, bez dodatnih (nepotrebnih) opcija koje umnogogme zbunjuju investitora i nadzora prilikom samog sprovođenja ispitivanja šipova. S obzirom na iskustvo autora u ispitivanju integriteta i

5 CONCLUSIONS

The pile integrity and load testing methodology is defined by the procedures prescribed in the pile testing standards. The improvement of segments of the existing standards is reflected in: a more detailed clarification of the individual testing phases, the reduction and selection of the testing methods and procedures within a single test, since there is a larger number of testing options that are not used ordinarily in the existing testing standards, we present the key elements of pile integrity and load testing, without additional (unnecessary) options which, mostly confuse investors and supervision during the actual pile testing conduction. Considering our own experience in pile integrity and load testing, acquired nosivosti šipova, na nekoliko stotina, pa i hiljada ispitivanja, u ovom radu prikazane su metodologije ispitivanja koje su se najbolje i pokazale u praksi, i čiji je stepen pouzdanosti, s vremenom, dodatno usavršavan. Definisane su metode koje se koriste u analizi, obradi i interpretaciji podataka, što nije prikazano u izvornim standardima ispitivanja šipova.

Problematika ispitivanja integriteta i novosti šipova multidisciplinarnog je karaktera, jer zahteva integraciju znanja i iskustva iz oblasti: građevinarstva, geologije, geotehnike, analize i obrade signala, softverskog i hardverskog inženjerstva, tako da se konstantno javlja potreba za usklađivanjem gotovo svih aspekata ispitivanja. Posebno je osetljiva problematika u vezi s teorijom i obradom signala i hardverskim i softverskim inženerstvom, jer se moderne informacione tehnologije stalno unapređuju. U tom smislu, pred inženjere građevinarstva i geotehnike stalno se nameće pitanje usavršavanja iz ove oblasti, pri čemu, jednu od polaznih osnova informativno-edukativnog karaktera, može predstavljati i ovaj rad, u kom su, pored standardizovanih procedura ispitivanja šipova, definisana i usklađena iskustva i znanja autora iz većeg broja različitih metoda ispitivanja šipova.

through several hundred, and even thousands of tests, this paper presents the testing methodologies that have stood out in practice, and whose degree of reliability, has been further enhanced through time. The methods used in data analysis, processing and interpretation are defined, and this hasn’t been presented in the original pile testing standards.

The problem of pile integrity and load testing is multidisciplinary in nature, since it requires the integration of knowledge and experience in the fields of civil engineering, geology, geotechnics, signal analysis and processing, software and hardware engineering, so there is a constant need for harmonisation of almost all testing aspects. The issues of theory and signal processing, and hardware and software engineering are particularly sensitive, because of the constant improvements in modern information technologies. In this regard, the civil and geotechnical engineers are constantly faced with the question of improvement in this field, where this paper can be one of the informative and educational starting points, and in which, beside the standardised pile testing procedures, we have defined and matched personal experiences and knowledge from a number of different pile testing methods.

ZAHVALNICA

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REZIME

ISPITIVANJE INTEGRITETA I NOSIVOSTI ŠIPOVA: METODOLOGIJA I KLASIFIKACIJA

Mladen ĆOSIĆ
Kristina BOŽIĆ-TOMIĆ
Nenad ŠUŠIĆ

U radu su prikazane metodologija i klasifikacija ispitivanja integriteta i nosivosti šipova, u saglasnosti sa važećim inostranim standardima, ali i sa sopstvenim definisanim segmentima unapređivanja standarda i sopstvenim definicijama određenih ključnih elemenata. Klasifikacija je sprovedena po tipovima testova u kojima su jasno definisani: tok ispitivanja šipova, metode analize i obrade rezultata ispitivanja. Pored osnovne podele testova ispitivanja šipova na testove integriteta i testove nosivosti, dodatno je definisana i grupa testova kontrole bušotine kod bušenih šipova, s obzirom na to što je za pravilno formiranje bušotine, kada je reč o bušenim šipovima, neophodno prethodno ispuniti određene kvalitativno-kvantitativne kriterijume. Ovako prikazane metodologija i klasifikacija ispitivanja integriteta i nosivosti šipova prvenstveno služe za edukativne svrhe inženjera građevine i geotehnike koji se bave ovom problematikom, da dodatno donese novine na ovom polju ispitivanja i da dodatno pojasne sve elemente ispitivanja, budući da se u praksi vrlo često susreću protivrečna mišljenja i nesuglasice oko detalja ispitivanja.

Ključne reči: šip, ispitivanje, standardi, klasifikacija, integritet, nosivost

APSTRACT

PILE INTEGRITY AND LOAD TESTING: METHODOLOGY AND CLASSIFICATION

Mladen COSIC
Kristina BOZIC-TOMIC
Nenad SUSIC

The paper presents the methodology and classification of pile integrity and load testing, in compliance with current foreign standards, as well as our own defined segments of standard improvement and our own definitions of certain key elements. The classification has been conducted according to the test types which clearly define the pile testing process, analysis methods, and test results processing. Beside the basic division of pile testing to integrity tests and load tests, there is also an additionally defined group of shaft control tests in the case of bored piles, since for the proper shaft formation, when it comes to bored piles, certain qualitative-quantitative criteria must be fulfilled beforehand. Presented in this way, the methodology and classification of pile integrity and load tests serves, primarily, an educational purpose for civil and geotechnics engineers who deal with this issue, to additionally introduce innovations in this field of testing and clarify all the elements of the testing since contradictory opinions and disagreements regarding the testing details are quite common in practice.

Key words: pile, testing, standards, classification, integrity, load

UPUTSTVO AUTORIMA*

Prihvatanje radova i vrste priloga

U časopisu Materijli i konstrukcije štampaće se neobjavljeni radovi ili članci i konferencijska saopštenja sa određenim dopunama, iz oblasti građevinarstva i srodnih disciplina (geodezija i arhitektura). Vrste priloga autora i saradnika koji će se štampati su: originalni naučni radovi, prethodna saopštenja, pregledni radovi, stručni radovi, prikazi objekata i iskustava (studija slučaja), kao i diskusije povodom objavljenih radova.

Originalni naučni rad je primarni izvor naučnih informacija i novih ideja i saznanja kao rezultat izvornih istraživanja uz primenu adekvatnih naučnih metoda. Dobijeni rezultati se izlažu sažeto, ali tako da poznavalac problema može proceniti rezultate eksperimentalnih ili teorijsko numeričkih analiza, tako da se istraživanje može ponoviti i pri tome dobiti iste ili rezultate u okvirima dopuštenih odstupanja, kako se to u radu navodi.

Prethodno saopštenje sadrži prva kratka obaveštenja o rezultatima istraživanja ali bez podrobnih objašnjenja, tj. kraće je od originalnog naučnog rada.

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Stručni rad predstavlja koristan prilog u kome se iznose poznate spoznaje koje doprinose širenju znanja i prilagođavanja rezultata izvornih istraživanja potrebama teorije i prakse.

Ostali prilozi su prikazi objekata, tj. njihove konstrukcije i iskustava-primeri u građenju i primeni različitih materijala (studije slučaja).

Da bi se ubrzao postupak prihvatanja radova za publikovanje, potrebno je da autori uvažavaju Uputstva za pripremu radova koja su navedena u daljem tekstu.

Uputstva za pripremu rukopisa

Rukopis otkucati jednostrano na listovima A-4 sa marginama od 31 mm (gore i dole) a 20 mm (levo i desno), u Wordu fontom Arial sa 12 pt. Potrebno je uz jednu kopiju svih delova rada i priloga, dostaviti i elektronsku verziju na navedene E-mail adrese, ili na CD-u. Autor je obavezan da čuva jednu kopiju rukopisa kod sebe.

Od broja 1/2010, prema odluci Upravnog odbora Društva i Redakcionog odbora, radovi sa pozitivnim recenzijama i prihvaćeni za štampu, publikovaće se na srpskom i engleskom jeziku, a za inostrane autore na engleskom (izuzev autora sa govornog područja srpskog I hrvatskog jezika).

Svaka stranica treba da bude numerisana, a optimalni obim članka na jednom jeziku, je oko 16 stranica (30000 slovnih mesta) uključujući slike, fotografije, tabele i popis literature. Za radove većeg obima potrebna je saglasnost Redakcionog odbora.

* U**putstvo autorima je modifikovano i treba ga, u pripremi radova, slediti.**

GUIDELINES TO AUTHORS

Acceptance and types of contributions

The Building Materials and Structures journal will publish unpublished papers, articles and conference reports with modifications in the field of Civil Engineering and similar areas (Geodesy and Architecture).The following types of contributions will be published: original scientific papers, preliminary reports, review papers, professional papers, objects describe / presentations and experiences (case studies), as well as discussions on published papers.

Original scientific paper is the primary source of scientific information and new ideas and insights as a result of original research using appropriate scientific methods. The achieved results are presented briefly, but in a way to enable proficient readers to assess the results of experimental or theoretical numerical analyses, so that the research can be repeated and yield with the same or results within the limits of tolerable deviations, as stated in the paper.

Preliminary report contains the first short notifications on the results of research but without detailed explanation, i.e. it is shorter than the original scientific paper.

Review paper is a scientific work that presents the state of science in a particular area as a result of analysis, review and comments, and conclusions of published papers, on which the necessary data are presented clearly and critically, including the own papers. Any reference units used in the analysis of the topic are indicated, as well as papers that may contribute to the results of further research. If the reference data are methodically systematized, but not analyzed and discussed, such review papers are classified as technical papers.

Technical paper is a useful contribution which outlines the known insights that contribute to the dissemination of knowledge and adaptation of the results of original research to the needs of theory and practice.

Other contributions are presentations of objects, i.e. their structures and experiences (examples) in the construction and application of various materials (case studies).

In order to speed up the acceptance of papers for publication, authors need to take into account the Instructions for the preparation of papers which can be found in the text below.

Instructions for writing manuscripts

The manuscript should be typed one-sided on A-4 sheets with margins of 31 mm (top and bottom) and 20 mm (left and right) in Word, font Arial 12 pt. The entire paper should be submitted also in electronic format to e-mail address provided here, or on CD. The author is obliged to keep one copy of the manuscript.

As of issue 1/2010, in line with the decision of the **Management Board of the Society and the Board of Editors, papers with positive reviews, accepted for publication, will be published in Serbian and English, and in English for foreign authors (except for authors coming from the Serbian and Croatian speaking area).**

Each page should be numbered, and the optimal length of the paper in one language is about 16 pages (30.000 characters) including pictures, images, tables and references. Larger scale works require the approval of the Board of Editors.

Naslov rada treba sa što manje reči (poželjno osam, a najviše do jedanaeset) da opiše sadržaj članka. U naslovu ne koristiti skraćenice ni formule. U radu se iza naslova daju ime i prezime autora, a titule i zvanja, kao i ime institucije u podnožnoj napomeni. Autor za kontakt daje telefon, adresu elektronske pošte I poštansku adresu.

Uz sažetak (rezime) od oko 150-250 na srpskom i engleskom jeziku daju se ključne reči (do sedam). To je jezgrovit prikaz celog članka i čitaocima omogućuje uvid u njegove bitne elemente.

Rukopis se deli na poglavlja i potpoglovlja uz numeraciju, po hijerarhiji, arapskim brojevima. Svaki rad ima uvod, sadržinu rada sa rezultatima, analizom i zaključcima. Na kraju rada se daje popis literature.

Kod svih dimenzionalnih veličina obavezna je primena međunarodnih SI mernih jedinica.

Formule i jednačine treba pisati pažljivo vodeći računa o indeksima i eksponentima. Autori uz izraze u tekstu definšu simbole redom kako se pojavljuju, ali se može dati i posebna lista simbola u prilogu.

Prilozi (tabele, grafikoni, sheme i fotografije) rade se u crno-beloj tehnici, u formatu koji obezbeđuje da pri smanjenju na razmere za štampu, po širini jedan do dva stupca (8 cm ili 16,5 cm), a po visini najviše 24,5 cm, ostanu jasni i čitljivi, tj. da veličine slova i brojeva budu najmanje 1,5 mm. Originalni crteži treba da budu kvalitetni i u potpunosti pripremljeni za presnimavanje. Mogu biti i dobre, oštre i kontrastne fotokopije. Koristiti fotogrfije, u crno-beloj tehnici, na kvalitetnoj hartiji sa oštrim konturama, koje omogućuju jasnu reprodukciju.

U popisu literature na kraju rada daju se samo oni radovi koji se pominju u tekstu. Citirane radove treba prikazati po abecednom redu prezimena prvog autora. Literaturu u tekstu označiti arapskim brojevima u uglastim zagradama, kako se navodi i u Popisu citirane literature, napr [1]. Svaki citat u tekstu mora se naći u Popisu citirane literature i obrnuto svaki podatak iz Popisa se mora citirati u tekstu.

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The abstract (summary) of about 150-250 words in Serbian and English should be followed by key words (up to seven). This is a concise presentation of the entire article and provides the readers with insight into the essential elements of the paper.

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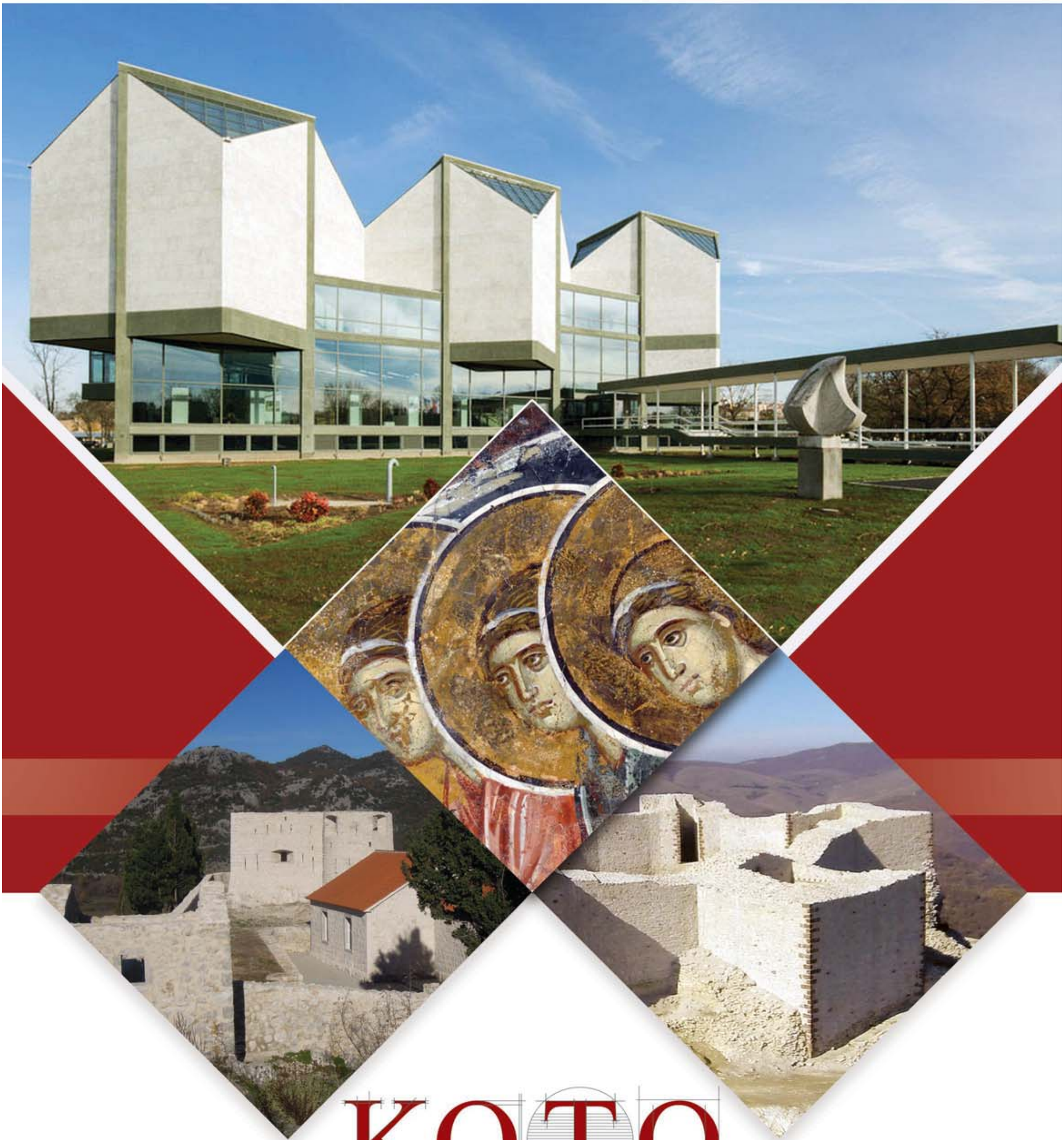

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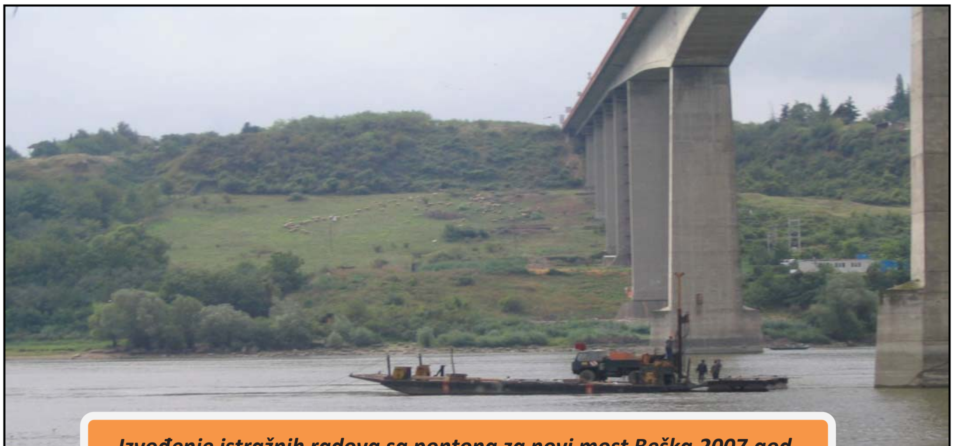


Izvođenje istražnih radova sa pontona za novi most Beška,2007.god.

Geotehnička istraživanja i ispitivanja – in situ

Od terenskih istražnih radova izdvajamo izvođenje istražnih bušotina (IB), standardnih penetracionih opita (SPT), statičkih penetracionih opita (CPT i CPTU), opita dilatometarskom sondom (DMT i SDMT), ispitivanja vodopropustljivosti tla različitim terenskim metodama (VDP), ugradnja pijezometara i dr.
Terenske metode ispitivanja šipova zauzimaju značajno mesto u našoj delatnosti, a na tržištu se izdvajamo kao lideri u toj oblasti u protekloj deceniji.

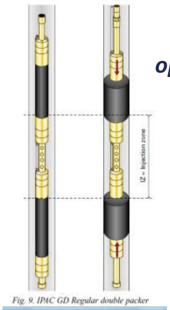

DLT-dinamičko ispitivanje šipova

Ispitivanje šipova

SLT metoda (Static load test) ispitivanje nosivosti šipova statičkim opterećenjem;
DLT metoda (Dynamic load test) ispitivanje nosivosti šipova dinamičkim opterećenjem;
PDA metoda (Pile driving analysis) omogućava praćenje i optimizaciju procesa pobijanja prefabrikovanih betonskih i čeličnih šipova u tlo;
PIT (SIT) metoda (Pile(Sonic) integrity testing) koristi se za ispitivanje integriteta izvedenih šipova (dužine, prekida, suženja ili proširenja).


CPT/CPTU opiti


Aktivno klizište



oprema za ispitivanje vodopropusnosti stena pod pritiskom do 10 bar-a metodom LIŽONA

Laboratorija za puteve i geotehniku

Laboratorija za puteve i geotehniku akreditovana je kod Akreditacionog tela Srbije – ATS prema SRPS ISO/IEC 17025:2006. U njoj se vrše ispitivanja tla (identifikaciono–klasifikaciona ispitivanja, fizičko–mehanička modelska ispitivanja), kamenog agregata i brašna, bitumena i bitumenskih emulzija, asfaltnih mešavina. U okviru laboratorijskih ispitivanja na terenu vrši se kontrola kvaliteta ugrađenog materijala i izvedenih radova (prethodna, tekuća, kontrolna ispitivanja i izvođenja opita in situ).

Projektovanje puteva i sanacija klizišta

U okviru projektovanja značajno mesto u radu zauzimaju geotehnička istraživanja terena i projekti sanacije klizišta - nestabilnih kosina useka i nasipa puteva i prirodno nestabilnih padina . Značajna su i projekovanja svih vrsta fundiranja specijalnih geotehničkih konstrukcija. Ističe se i iskustvo u oblasti putarstva, na projektovanju novih, rehabilitacija i rekonstrukcija postojećih puteva svih rangova sa pratećim objektima i dimenzionisanjem kolovoznih konstrukcija.

Nadzor

Naši inženjeri imaju veliko iskustvo u kontroli i proveri kvaliteta izvođenja svih vrsta radova, kontroli građevinske dokumentacije i praćenju radova u skladu sa njom, kao i rešavanju novonastalih situacija tokom izvođenja radova.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- Najbolja tehnička ekspertiza i praksa za sanaciju betona i strukturalna ojačanja
- Odlična reputacija kod vodećih izvođača i ugovarača posla

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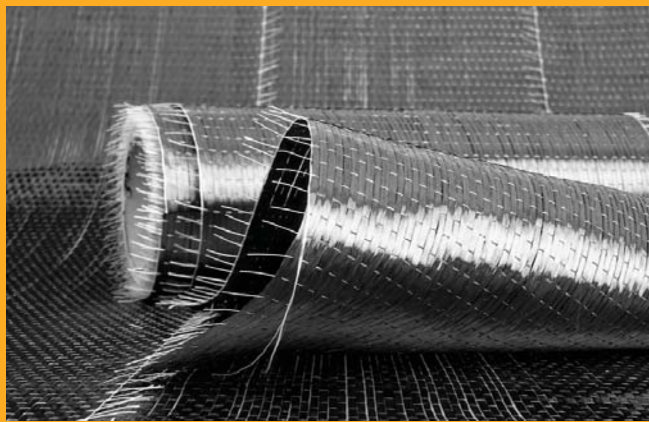

- Integrisani proizvodi i sistemi visokih performansi koji mogu da povećaju i poboljšaju kapacitet, efikasnost, trajnost i estetiku zgrada i drugih objekata – u korist naših klijenata i boljeg održivog razvoja
- Sika mreža obučenih i iskusnih građevinskih stručnjaka

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- Posebna rešenja završnih ojačanja za korišćenje kod betona slabije jačine i drugih podloga

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