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Iron Load-bearing Structure and Formal Characteristics in the Nineteenth Century Historicist Architecture

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Abstract

This study analyzes the role played by load-bearing structures in determining the formal characteristics that distinguish the historicist architecture of the nineteenth century that uses iron load-bearing structures or components.
As shown in the examples studied, except for some proportion or size changes, iron structures did not produce significant changes at the level of architectural space and form, beeing in most cases subject to the aesthetic rules imposed by established architectural language. In those few situations where the load-bearing system has brought significant changes, this issue was managed distinctively by historicist architecture: 1 - in an attempt to provide prestige and overall coherence, it has used the morphology of the established architectural language in order to subordinate through decoration; 2 - through new principles of order, derived from those that formed the basis of established architectural language, some of which later become paradigmatic in the modern architecture, it has provided the conceptual framework through which new typologies could be born.
The conclusion underlines that the iron load-bearing structures by them self were not able to determine any formal characteristics of the historicist architecture. Their role was that of a catalyst for change, opening up the established architectural language.

Rezumat

În cadrul acestui studiu s-a analizat rolul pe care structura portantă l-a avut în determinarea caracteristicilor formale care diferenţiază arhitectura istoristă care utilizează structuri sau componente structurale metalice.
Aşa cum rezultă din exemplele studiate, în cele mai multe cazuri, dincolo de eventuale modificări de proporţie sau de gabarit, structura metalică, supusă regulilor impuse de limbajul arhitectural consacrat, nu a avut forţa de a produce schimbări esenţiale la nivelul spaţiului şi formei. În acele situaţii în care sistemul structural a adus schimbări importante, mai ales în ceea ce priveşte spaţialitatea clădirii, problema a fost gestionată distinct de arhitectura istoristă: 1 - în încercarea de a oferi prestigiu şi coerenţă ansamblului, limbajul arhitectural consacrat a căutat să subordoneze prin decoraţie; 2 - noi principii de ordine, derivate din cele care au stat la baza limbajului arhitectural consacrat, şi care au devenit paradigmatice mai târziu în arhitectura modernă, au oferit cadrul conceptual în care s-au putut naşte noi tipologii.
Concluziile arată că structura metalică nu a determinat în sine caracteristici formale în arhitectura istoristă. Rolul ei a fost unul de catalizator al schimbării limbajului arhitectural consacrat.

Keywords: load-bearing structure, iron, historicist architecture, formal characteristics, catalyst for change

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1. Introduction

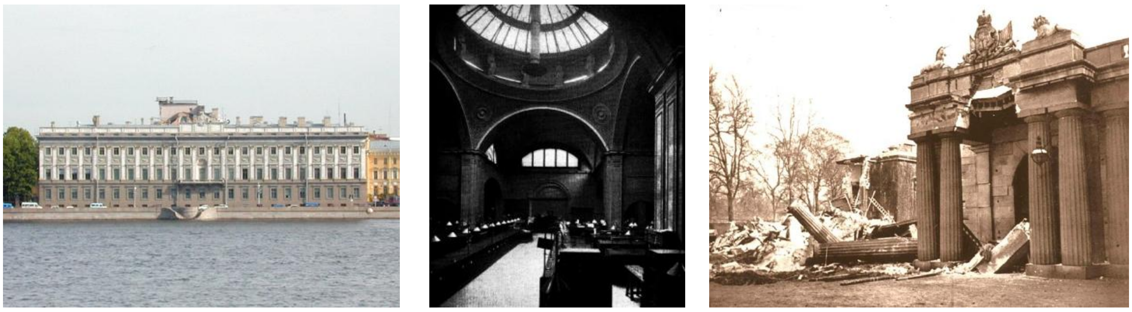
With the development of industrial processes and production techniques for obtaining cast iron, by the beginning of the nineteenth century, the amount of iron available in the construction market has growth considerable thus increasing its importance as a viable building material. Until the eighteenth century, one could speak of iron as nothing more than a material used for auxiliary structural elements, without any major influence on architectural design philosophy. However the rapid growth of iron production would bring it to the forefront, as a structural material worthy of use from the first stages of design, challenging through its undisputed qualities and opportunities the traditional ones like stone and wood.
However, especially within the field of public works, the mainstream architecture of the time responded to the excesses of the baroque and rococo through a recuperation of the classical language and morphology. Going through the examples offered by the history of architecture, it is easy to note that most significant and spectacular changes in architectural space and form would initially appear outside these historicist trends. Made possible mainly by the development of commercial buildings whose form showed little concern with stylistic elaboration, these changes came as pragmatic responses to the new requirements imposed by industrialization. We include here those buildings needed for production, storage, transport and trade, united by some authors under the name "architecture of the engineers" (Vasilescu, 1989)[1], which will proove to be extremely influencial on the architecture of the following centuries. The goal of this study will be to put some light on the means through which, the development of the iron load-bearing structures, promoted by changes in the utilitarian buildings, has come to affect mainstream, historicist architecture of the nineteenth century. Furthermore we will identify the role that the load-bearing structure occupied in determining formal and structural characteristics which distinguished certain examples between the buildings of the historicist styles.

2. Neoclassical Architecture

Neoclassical style, already defined years before the abundance of iron in the construction market, began as a purist movement in search of intellectual truth. It represented in terms of style a manifestation against Baroque architecture, which was considered 'untrue' because of the emphasis on illusion and applied ornament [2]. Using over time various classical sources as stylistic inspiration, sources that had been considered 'true' (the purity of Roman art, the ideal of the ancient Greece and later the Renaissance architecture) but finally accepting as 'true' source even the contested Baroque architecture, the Neoclassical architecture would become the language of the École des Beaux-Arts, representing the paradigmatic approach for most public buildings of the nineteenth century. Characteristic to a very long period, this style continues with various stages that are intertwined in the historicist eclectic manifests of the early twentieth century that has culminated in the Belle Epoche and in the periods of dictatorship started before the Second World War [3]. Considered in the late nineteenth century anti-modern or even reactionary [3] - although rigorous, simple volumes and innovative use of spatial effects come to impose it, by several prominent representatives as Boullee, Ledoux and Schinkel as source of inspiration to modern architecture - Neoclassical architecture has, most often, incorporated or dressed in a decorative language, the new construction material in question. Since the theory on which it was based was founded on established building solutions, taking into account a relationship between load-bearing structure and architectural form, joint detailing and applied ornament[1], in neoclassical architecture iron was not considered a necessary building material. Within this architectural language, cast iron

[1] the load-bearing structure, based on compression resistant materials that carries load trough mass, is part of the determining force in the global form; the jointing detail has a strong relationship with the applied ornament, this having the role of articulating and emphasizing the structural components

or wrought iron load-bearing components correctly sized, would not produce the right and required proportion, being too flimsy against the bulkiness of the massive stone carved classical orders. However, some of the most important buildings attributed to this style contain load-bearing components of cast or wrought iron. In such buildings iron, taken as an alternative, replaces conventional materials for various reasons, highlighting the material versatility. Fireproofing requirements imposed it in some roof or floor load-bearing structures. Simple economic reasons brings classically shaped iron columns - casting columns with classical decorative moldings proving less expensive than carving them. It's high resistance and rigidity made it the material of choice for beams dedicated to take over large spans, aswell as for more slender columns thus freeing up the space. No less important is the use of iron in neoclassical architecture in masonry reinforcement. The effect was felt in the size and proportion of the masonry load-bearing elements. Commercial buildings are among the few situations where the utilitarian iron load-bearing structures are adjoined or enclosed by neoclassical walls. As an exception to the rule we find Henry Labrouste's[2] Bibliothèque Sainte-Geneviève and Bibliothèque Nationale as two absolutely remarkable examples commissioned in the the field of public buildings. Labrouste, who would later become the symbol of structural rationalism was, as Giedion argues, „... a man who unites the ability of both the engineer and the architect ..." (1941 :218)[5]. Such examples already represent certain models of exploiting both, the expressive qualities and the load-bearing potential of the iron structures.



a *b* *c*

Figure 1. a - Marble Palace; b - Bank Stock Office; c - Buckingham Palace (North Lodge).

The Marble Palace (1768-72) in St. Petersburg, conceived by the architect Antonio Rinaldi *Fig.1-a* in early neoclassical style, is one of the first buildings to use iron beams (Hitchcook, 1958: 116)[6]. In 1779-1781 one of the first representatives of the French neoclassicism, "... the very technically minded architect of the Paris Pantheon ...", Jacques Germain Soufflot, has used an iron roof structure over the stair-hall leading up to the Grande Galerie of the Louvre, Paris (idem)[6]. „Horrified by the reccurent fires at Palais Royal ...", the new roof designed by Victor Louis, the architect of the new French Theatre (1786-1790), uses some principles developed by two "... rather obscure French architects ... " Ango and Eustache Saint-Fart - iron frame and ceramic hollow tiles (idem)[6]. Sir John Soane, one of the revolutionary innovators of the British neoclassicism, avoids the use of wood in the fireproof vaults of the Bank Stock Office, London 1794, using also ceramic elements set within an iron frame (idem: 117)[6]. Alone the oculus in the central dome, covered with iron and glass *Fig.1-b*, allows the observer to read some of the metal structural components. At the Buckingham Palace in London (1825-30) the columns on the north wing are cast to the classical proportion [7]. The amorphous metal submits to the normative language imposed by the Doric style design of the architect John Nash *Fig.1-c*. Given the need to maintain a fluid space, in order to support the gallery of the main hall of the Royal High School, Edinburgh, 1825-9, the

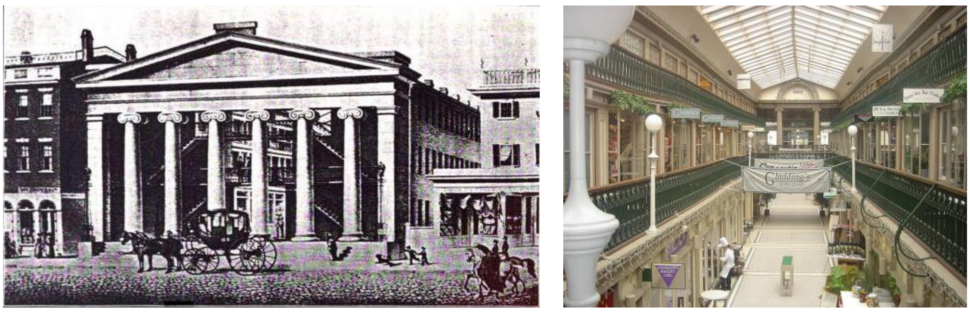
[2] Pierre François Henri Labrouste (1801-1875), architect, alumni of the École des Beaux Arts, winner of the Grand Prix de Rome din 1824. "influenced by E.-J. Gilbert, whose training as an engineer as well as an architect freed him from reliance on heavy academicism." [4]

architect Thomas Hamilton defines a new interior spatiality using unusually proportioned cast iron columns with floral decorated capitals (Mignot, 1983: 39)[8]. Faced with the conflicting situation presented by the need to support the gallery while at the same time maintaining the continuity of the space, the architect has used iron as the only material capable of producing columns slender enough to respond to both, functional and structural requirements.

a *b* *c*

Figure 2. a - King's Library; b - St. Isaac; c - British Museum reading room.

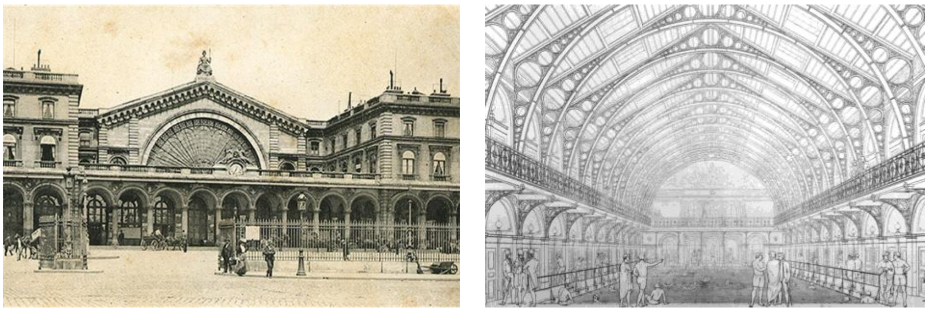
The extraordinary dimensions (91m length, 12m height, 9m width, with a central section of 18m) required by the space of the King's Library 1823-7 *Fig.2-a*, in the British Museum imposed the use of cast iron beams [9]. The very idea of designing such a space could not be uttered without the capabilities of the newly discovered material. While exploiting the freedom to choose larger openings, architect Robert Smirke does not feel the need to betray, or even highlight the solution that allows such a performance. Although it represents a remarkable technical innovation, enabling an absolutely exceptional opening, comparable with the great openings of the cupolas in the Renaissance and Baroque, without the efforts implied by the construction of a masonry dome, the cast iron skeleton proposed by the architect August Augustovich Monferan for the dome of the St. Isaac Cathedral, erected in St. Petersburg in 1842 (Hitchcook, 1958: 116)[6], remains also hidden behind the classical scenery *Fig.2-b*. A similar position in respect to the iron structure can be observed with the covering of the new reading room in the British Museum *Fig 2-c*, conceived in 1857 by Sidney Smirke (Hitchcook, 1958: 127)[6].



a *b*

Figure 3. a - Westminster Arcade, b - atrium.

At the beginning of the nineteenth century, the shopping galleries introduced a new building typology. The spatiality of such a building was strongly linked to the expressive and structural potential of iron structures. Besides providing protection from the weather elements, the specific iron and glass roof was the only one that could provide the necessary amount of light for the main space, represented by the atrium, with its multilevel galleries that assures the large surfaces required for the modern presentation of the commodities. At the Westminster Arcade in Providence, Rhode Island *Fig.3-a,b*, one of the oldest galleries in the United States, designed by the architects Russel Warren and James Bucklin in 1828, as a great temple of commerce (Smith, 1996: 172)[10], we have an example of such a building being treated with the aesthetic consideration deserved by only the most important public edifices. The early neoclassical style, that organizes the masonry construction that encloses the great atrium is intended here to provide prestige to this new building program.



a *b*

Figure 4. a - Gare de l'Est; b - Dianabad.

In a context in which the stylistic concern was to rather mask "... the success with which new functional needs were satisfied in this period by the bold use of the new materials and new types of construction." (Hitchcook, 1958: 76)[6], Gare de l'Est *Fig.4-a*, built in Paris between 1847-52 after the drawings of architect François Duquesney, as a major monument of the Classical rationalism, serves as a reference for this new critical approach. Here, the facade chooses to express some of the spatiality of the utilitarian interior marked by the great iron and glass roof that cover the rails. Offering shelter to another novel function, the first covered pool on the European continent, Dianabad in Vienna *Fig.4-b*, conceived by the architect Ludwig Förster together with the architect-engineer Karl Etzel and erected between 1841-3, also presents an elegant cast iron roof, sincerely exposed in the interior. "... the circular bracing of the iron principals, a frequent motif in large openwork members of cast iron at this time, was most appropriate to the Rundbogenstil detailing of the masonry walls."(Hitchcook, 1958: 123)[6].

a *b* *c*

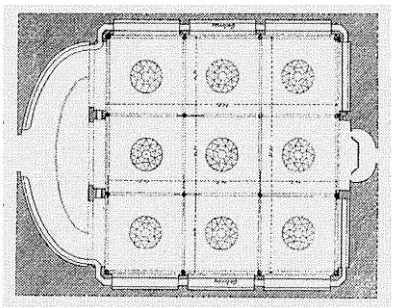
Figure 5. a - Bauakademie; b- Bibliothèque Sainte-Geneviève exterior view, c - reading room.

Another perspective on the classical norms is offered by the Bauakademie of Berlin *Fig.5-a*, remarkable, among others, for the innovative approach, almost completely devoid of applied ornament. Critically interpreted, classical language is resumed here to the ordering of the construction. Erected between 1831-6, this seminal work by Karl Friedrich Schinkel[3] includes iron beams that support brick arches in a system, that the architect studied in England (Darley, 2003: 26-30)[12], while researching the technologies of the industrial revolution. The load-bearing system, with iron beams supported on masonry piles, designed in collaboration with the engineer Peter Beuth (Adam, 2004: 11)[13], is considered one of the precursors of the skeletal load-bearing system. The Sainte-Geneviève library in Paris *Fig.5-b,c*, built between 1843-50 after the plans conceived by Henry Labrouste in the years 1839-42 is one of the first public buildings that uses

3 Karl Friedrich Schinkel (1781-1841) one of the most influential architects of the nineteenth century. His work represents an important source of inspiration for the architects of the first half of the XX century. [11]

„openly and extensively" an iron load-bearing structure (Hanser, 2006: 34-38)[14]. Radically different from the conventional solutions, this library counts on the principles on which the English factories were built: the Neorenaissance masonry, avoiding historical references, integrate „... like the works of a watch in its case." (Giedion, 1941: 220)[5], an elegant iron structure with arcs and columns, that could be considered having Gothic influences (Blanc, 1993: 6)[15]. This answer came as the result of a rational thinking, in them functional requirements have been those who imposed the approach. Much like in industrial buildings, nothing seems here to be arbitrary.

Sober to the point of austerity-there is almost no decoration - its facade is neither pretty nor even elegant. There are no classical orders and no reference to any buildings from the historical past. To many contemporaries, the building had no style. Labrouste wanted its functions and its real, undisguised structure to order his building. Any ornament was to derive from the latter and help the public understand the former. They should be able to read his building "like a book". (Hanser, 2006: 36)[14]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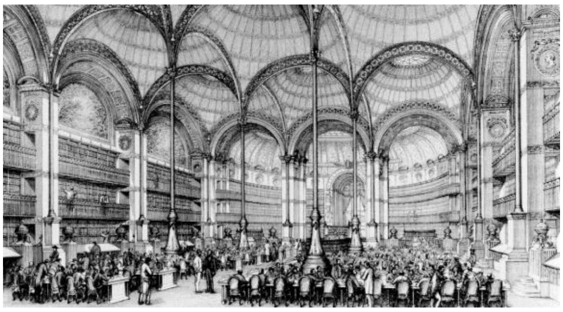
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Figure 6. a - Bibliothèque Nationale reading room plan, b - interior view.

Built between 1862-1868, the reading room of the Bibliothèque Nationale in Paris *Fig.6-a,b*, would already represent an important technical advance (Hitchcook, 1958: 128)[6]. The design of Labrouste proposes eight sky-lighted terracotta domes carried by wrought-iron arches and slender cast iron columns, in a load-bearing system that will amaze (or confuse) its contemporaries trough its spatiality, elegance and functionality. The solution offered by Labrouste for the central book depository, as an answer to the increasing needs of storage space imposed by the book production of the nineteenth century, certainly represents another masterpiece. The use of the cast iron grid plates for the floor construction, characteristic maybe for the engine rooms of the steamships, unveils "the germ of new artistic possibilities" (Giedion, 1941: 226)[5].

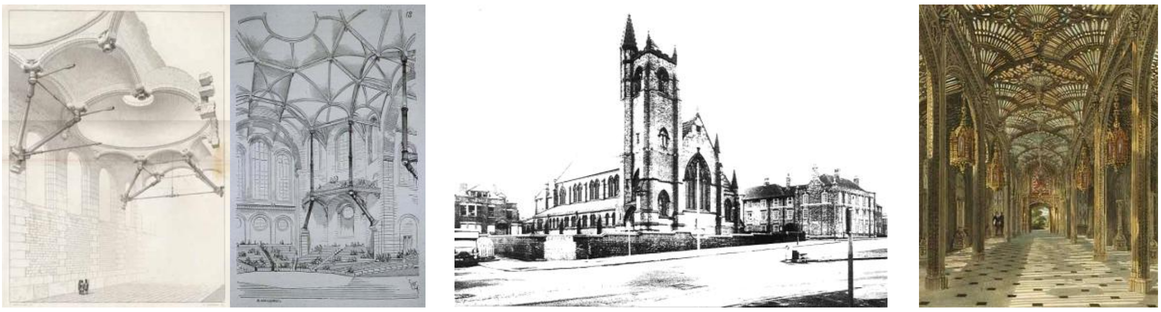
3. Neogothic Architecture

The use of large scale iron load-bearing elements has it's origin at the beginning of the Neogothic style in architecture. This period, dominated by sentimentalism and the picturesque, less attractive in terms of style, was defined by the abundance of cast iron on the building material market. This availability has thus encouraged the search for the most diverse applications. The Gothic pillars, the Gothic ornaments, originally carved in stone, found their cheap substitute in iron elements that could be easily replicated by casting. The first Gothic iron load-bearing structures constructed in this period, represent an ideological irony. Through their industrialized production techniques they were in utter contradiction with the romantic medieval revival theories that prophesied a return to an idyllic agrarian craft based society. Superficially, this doctrinal interpretation has led inevitably to the rejection of iron as a building material suitable for Gothic edifices. More profound, this principle emphasizing the primacy of the construction methods[4] rather than the image obtained by

[4] the construction methods, here medieval, considered to be the result of a healthy social organization, provided the basis, the forms resulted implicitly

simply copying ornamental details[5], would become referential in the decisive step towards the modern utilization of functionally determined iron parts in slender, exposed structural systems.. This principle has opened the horizon to the first attempts to adapt the formal language to the new needs and new means of building. In fact, the rationalist thinking, introduced with the systematic archaeological research that begins in the early nineteenth century, in contrast with the romantic origins, brings an essential contribution in the use and development of skeletal iron and steel load-bearing systems: encouraging freedom of expression and structural honesty, ideals that may be considered the essential engines of the modern movement. Even if the claims of representation were fundamentally different, the great structures of the mid nineteenth century (Crystal Palace or The glass roof of the Oxford University Museum) can be considered both the result of this rationalist thinking.

Among the architects who have embraced this style, between them those in France that were all amateur archaeologists who had restored at least one Gothic building before dedicating to their own projects [2], Viollet-Le-Duc[6], one of their leading exponents, saw the architecture of the nineteenth century based on a rational construction and composition system, found in the Gothic style, without the imitative ornamental detail. His treaties (Entretiens sur l'architecture), published between 1863 and 1872, as a set of unrealized projects that combine iron with masonry construction *Fig.7-a,b*, would later become a major source of inspiration for modern architects.



a *b* *c* *d*

Figure 7. a,b - plates from Entretiens sur l'architecture; c - St. Anne's Church; d - Carlton House Conservatory.

Maybe the first example of using iron at „truly architectural scale" is offered by the columns that support the gallery of the St. Anne Neo-gotic church in Great Richmond street, Liverpool, *Fig.7-c* constructed between 1770-2 (Hitchcook 1958: 116)[6]. The Palace in Kew, designed by the architect James Wayatt for George III in a „Castellated style", includes already an iron skeleton structure (idem: 117)[6].

In a period in that greenhouses were constructed in masonry, with large glazed areas between stone pillars and opaque roofs, the Carlton House Conservatory in London 1811-12 *Fig.7-d*, designed by architect Thomas Hopper (Hitchcook, 1958: 117)[6] searches also to exploit, in a super-ornate Neo-gotic version, the potential of the iron load-bearing structures. Even if the Gothic form completely subordinates the form of the load-bearing elements, this building is not without the merits. Much like in the Gothic load-bearing structural philosophy of design this representative ensemble, shows the extraordinary capacity of cast iron to be used in unprecedented slender structural components.

The churches in Liverpool, conceived in Neogothic style by the architect Thomas Rickmann and the iron-master John Cragg, emphasize another advantage of iron load-bearing structures, that of prefabrication. The first church, St. George in Everton 1812-14 *Fig8-a*, constructed around a cast

5 the ornament, here Gothic, the Gothic forms generally, have been considered to be the result of a process with deep cultural roots, they could not be obtained others than recovering the cultural values that constituted the foundation of their birth, so purely formal imitation been rejected.

6 „the last great theorist in the world of architecture" John Summerson 'Viollet-le-Duc and the Rational Point of View', in: Summerson, *Heavenly Mansions and other essays on architecure* (1948), New York 1963, p 135 (retrieved from Kruft, 1985 :282)[16]

iron skeleton, impressive trough the slender proportion of the load-bearing members, became a model for St. Michael in Hamlet 1813-15 *Fig.8-b*. Reusing of the casting molds from St. George for the prefabrication of the load-bearing and decorative elements for St. Michael, will bring substantially cost reduction [17]. Cast iron, used initially only in the interior, takes at St. Michael every details possible, becoming to be expressed even at the facade level [18].

Kreuzberg Memorial in Berlin, built between 1818-21 *Fig.8-c*, after the design of Karl Friedrich Schinkel (Mignot, 1984: 42) [8], is an early example of the use of iron in the Neogothic architecture outside the British island. The cast iron imitates Gothic detailing in a purely formal way, Schinkel exploiting here only its capacity to submit to form, to take without any difficulties the most complex ornamental shapes.

Substantially influenced by the doctrinaire, anti-industrial writings[7] of Gothic-revival theorist Augustus Welby Northmore Pugin's[8], the building of the Westminster Palace in London 1840 *Fig.8-d*, designed by the architect Charles Barry, at which Pugin himself took part as the main detail designer, would solve the problem of fire resistance by relying, on iron load-bearing components for the roofs and floor construction elements. (Hitchcook, 1958: 122)[6]. Carefully hidden from the eye of the beholder, these solutions were in the fact the culmination of technological development at the time.

a *b* *c* *d*

Figure 8. a - St. George's Church; b - St. Michael's Church; c - Kreuzberg Memorial; d - Westminster Palace.

In France, "... a work of considerable scale and technical elaboration ... ", was the iron roof that protected the vaults of Chartres Cathedral *Fig.9-a*, conceived and constructed by C.J. Baron and Nicolas Martin between 1837-9 (Hitchcook 1958: 108)[6]. Although hidden for the common viewer, this solution would become the prototype for similar construction. One such example could be found at the Sainte-Clotilde Basilica *Fig.9-b*, whose construction started in 1846 after the design of architect Franz Christian Gau, realized in 1836. These seemly first Neo-gotic church built in Paris would be completed by the architect Théodore Ballu in 1857 (Hitchcook 1958: 108)[6] Impressive trough it's iron spires which foresee the opportunities offered by the use of the new material, this building ultimately fails much like the others to find alternatives for established formal patterns. Moreover, the questionable proportions, the "characterless" and "deadly mechanical" detailing constitute questionable realities, critically emphasized in Hitchcook's presentation (1958: 108)[6]. A great example, rather by it's dimension that places it between the highest spires in France (151m), is the iron spire of the Rouen Cathedral *Fig.9-c*, that replaces the old lead covered wood spire. This construction, having as source of inspiration the spire of the Salisbury Cathedral, started in 1848 by architect Jean-Antoine Alavoine and completed by architects Eugene Barthelemy and L.F. Desmarest in 1877, presents at most a fascinating open cast

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7 We talk here about *Contrasts* (1836) and *The True Principles of Pointed or Christian Architecture* (1841) writings that marks "... a point in architectural theory at witch non-architectural and non-aesthetic considerations gained the upper hand." (Kruft, 1985: 327)[16].

8 Augustus Welby Northmore Pugin (1812-52), architect, designer, theoretician, "... introduced a new polemical and ideological tone in the Gothic debate." (Kruft, 1985: 327)[16].

iron structure. The first church in Paris built on a structure almost entirely of iron seems to be Saint Eugene, 1854-5 *Fig.9-d*, designed by architect Louis-Auguste Boileau (Hitchcoock, 1958: 128)[6]. Without offering extraordinary architectural spaces or forms, at most confusing trough the unusual dimension of the Gothic members, such examples remain witnesses to the potential provided by the skeletal structures made of the new material. The huge importance of the Gothic ornament in the consciousness of the Christian era can be observed in the prefabricated churches that were exported worldwide in the early 1850's, whose industrial image is sweetened using a number of Gothic details.

a *b* *c* *d* *e*

Figure 9. a - Chartres Cathedral - roof; b - Sainte-Clotilde; c - Rouen Cathedral; d - Saint Eugene Church; e - Oxford University Museum.

One of the few critical alternatives of using iron load-bearing structure in Neogothic buildings is to be found in the iron and glass covered courtyard of the Oxford University Museum, 1855-60 *Fig.9-e*, constructed after the design provided by the architects Thomas Deane and Benjamin Woodward. The style, strongly influenced by John Ruskin[9] seems to be, as Hitchcook remarks, the possible answer to the question: "How would medieval builders have used structural iron had it bean readily available to them?" (1958: 176)[6]. Materialized under the supervision of iron-master E.A. Skidmore, after a huge failure, the first structure that mainly used wrought-iron was unable to support his own weight [20], this building stands out between so many structures constructed at the time without architectural control. Noteworthy is the articulated character of the structure and the wrought-iron ornament whose development seems to be made to meet production technology. As Hitchcook remarks (idem)[6], this building could be "the first echo in England of the theories of Viollet-Le-Duc".

4. Eclectic Architecture

Paradoxically or not, the Eclectic architecture would pay great attention to the new, technically innovative solutions. "Apparently obsessed [more than ever] with stylistic elaboration" (Hitchcook, 1958: 154)[6], with finding the national architectural language, the appropriate source of inspiration, architects were preoccupied to incorporate in their buildings whatever could better satisfy the needs that constituted the base of edification, including the latest technical advance. Among the stylistic components which diversifies their source off inspiration, non-discriminatory coming from every historical period, from ancient Egypt to Louis XVI, taken with or without discernment structural principles, archetypal forms, plans or merely ornaments, one could find iron structures expressed with great boldness. However, even if economic and functional considerations lead to consistent use of iron as a load-bearing solution, its exposure remains rather exceptional. In

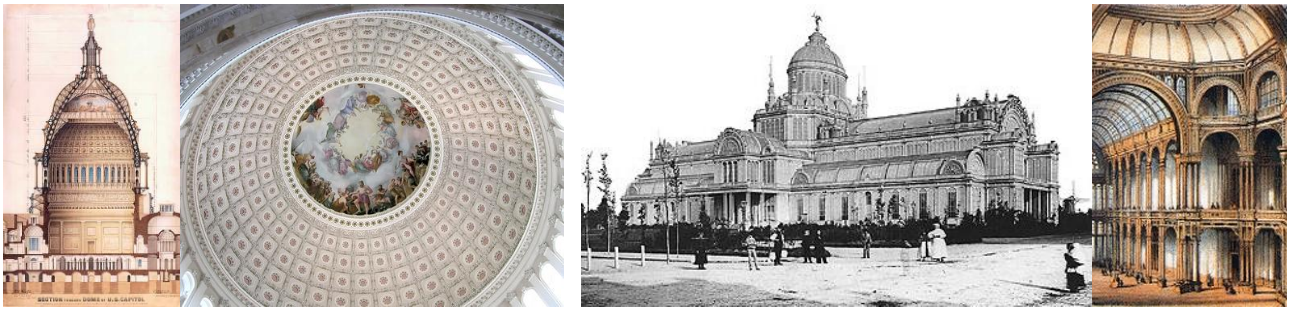
9 John Ruskin (1819-1900) - thinker, artist and art critic that significantly influenced the Arts and Crafts movement, known in architecture specially trough his writings *The Seven Lamps of Architecture* (1949) and *The stones of Venice* (1851-3). In his vision, "art is an expression of the values of a society", architecture being the most public of the arts. According to this point of view, architecture "most fully expresses the whole spirit of the people" [19].

most cases, buildings that house new architectural programs, in which the iron structures respond to the functional requirements, were treated with a decorative ambiguous language. In an attempt to express prestige, trough style, or to impose trough grandeur, the new material remains behind the scenes, supporting unseen, shapes whose foundation could not be found neither in the constructional system, nor in the stylistic dogmas that ordered the initial building composition. If this architectural style has not succeeded in finding the proper image for the new material or the new building systems, iron, in all its applications, has certainly been openly abused in the service of this style - most of the stylistic aberration that would be brought to light in this period would not have been possible without the contribution of the prefabrication industry, working in the service of the applied ornament [8].

a *b* *c*

Figure 10. a - Royal Pavilion; b - Coal Exchange exterior view, c - interior.

One of the examples of the romantic eclecticism, practiced before the period of glory, is the Royal Pavilion in Brighton, an early neoclassical building that was to be transformed in 1818-21 by the architect John Nash, which gives it an oriental „festive and frivolous" atmosphere, with Chinese and Indian influence (Hitchcook, 1958: 93-94,117)[6]. The kitchen and several of the rooms attached by Nash offer some of the first noteworthy examples for the use of iron in it's own scale, given the quality of the material, and not imitating masonry dimensions. Although the first sketches showed slender columns, without capitel, the ones which were put in place would be decorated with floral motifs to soften their visual impact *Fig.10-a*. Invisible, the load-bearing skeleton of the great bulb-shaped dome, is also made of iron (idem: 117)[6]. Another example of free use of iron load-bearing systems imposed by functional requirements is the Coal Exchange in London, built between 1846-9 after the drawings of architect James Bunsone Bunning. The "... two palazzo blocks set at a fairly sharp angle to one another and loosely linked by a very Picturesque round tower, free-standing in its upper stages ...", hide here an interior hard to guess: nearly invisible, the masonry leaves place to "... an elegant cage of iron elements rising to the glazed hemisphere above (Hitchcook, 1958: 123)[6] *Fig.10-b,c*. The US Capitol dome in Washington, built between 1855-1865 *Fig.11-a,b*, distinguished mainly by its size (Lee-Thorp, 2006: 103)[21], which can rival the greatest domes of Baroque in Europe, presents the typical hidden iron structure. A shape similar to that of Michelangelo's dome, decorated by the architect Thomas U. Walter in Second Empire style, hide an iron structure motivated by the ease of execution, low weight that could be supported by a pre-existing structure and reduced costs compared with those of a masonry dome. Paleis voor Volksvlijt in Amsterdam *Fig.11-c,d*, built in 1856 after the plans of architect Cornelis Outshoorn (Hitchcook, 1958: 126)[6] on the model of crystal palaces, presents a solution in which the iron structure again submits stylistically to the second Empire, the neo-Renaissance decoration seeking an 'improvement' of its expression.



a *b* *c* *d*

Figure 11. a - Washington Capitol section, b - interior view of the dome; c - Paleis voor Volksvlijt, d - interior.

Alexander Greek Thomson's churches, built in the second half of the nineteenth century, Vincent Street Church 1859 and Queen's Park Church 1867 *Fig.12-a* from Glasgow, also exposed iron on the inside. At Queen's Park, which combines the neoclassical style with a tower of Hindu influence, iron elements are used with a remarkable logic: “Both the heavy masonry tower - which is, of course, invisible from the interior - and the heavy clerestory are carried on these delicately proportioned metal columns with a frankness and boldness hardly equaled before the twentieth century.” (Hitchcook, 1958: 62)[6]. In the same period, Saint-Augustin Church in Paris *Fig.12-b*, built between 1860-7, a mixture of Romanesque, Byzantine and Italian Renaissance, that complements the image of the apartment buildings aligned to the Hausmanian boulevards, offered also an example, perhaps less inspired, of using iron load-bearing components. Hitchcook criticizes the way the architect Victor Baltard, who conceived Les Halles[10] in 1853, articulates the iron arches of the roof to the Romantic-Rennaissance design of the masonry (1958: 142)[6]. Also an example of exposed iron gives St. Mary's Church *Fig.12-c*, built between 1866-73 in the suburb of Ealing, after the Gothic-Byzantine design of architect Samuel Sanders Teulon (Hitchcook, 1958: 180)[6]. Here columns and arches support with great nonchalance the visible corrugated sheets that make up the roof covering.

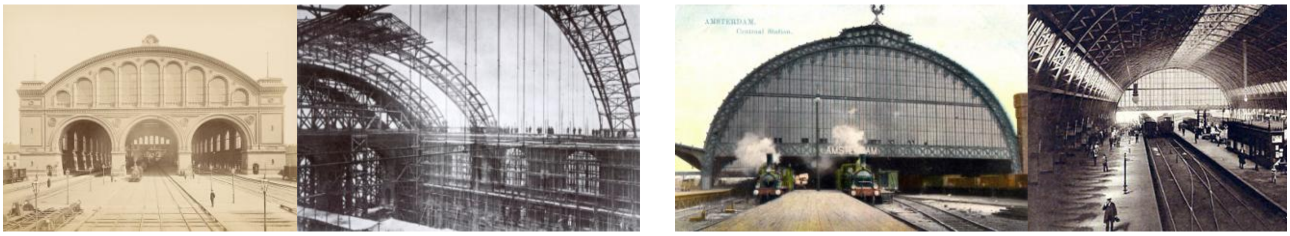
a *b* *c*

Figure 12. a - Queen's Park Church; b - Saint-Augustin Church; c - St. Mary's Church.

Whether it is classified as romantic or a rational phase of Neoclassicism (Hitckook, 1958: 27)[6], as a national eclectic style combining the rigor of classic romantic-medieval language, or just a version of the Neorenaissance (Collins, 1965: 98)[22] , the Rundbogenstil can be considered the result of adapting the architectural style to the new requirements and means. We speak about a style, whose flexibility, typical to the eclecticism, allows the architect, as Collins notes, “... to select - and even to invent for himself - such compositional and decorative forms as might be considered suitable for the occasion.”(1965: 98)[22]. As Hitchcook notes (1958: 154)[6], Rundbogenstil was to prove very well suited to order the composition dominated by the huge iron arched load-bearing structure of the Anhalter Bahnhof in Berlin (1872-80) *Fig.13-a,b*. Designed by architect Franz Schwechten,

[10] Les Halles Centrales (1853-8) - buildings in iron and glass designed by the architect Victor Baltard, which covered for more than a century the traditional central market in Paris

this station was a major step forward in clarity and coherence of functional organization. Equally impressive through the 62m span of the platform roof, the largest on the continent at that time (Hitchcook, 1954: 154)[6], designed by engineer Heinrich Seidel, Anhalter Bahnhof gives us a synthesis, already typical at the time, of the cooperation between the formal and technical field of specialization (Zietz, 1999: 15)[23]. A somewhat similar style will be found at Amsterdam Central Station (1881-9) *Fig.13-a,b*, designed by architects Pierre Cuyper and A.L. van Gendt, which incorporate the platform covering structure designed by the engineer I.J. Eijmer and built by iron-master Andrew Handyside from Derby, England. Even if the issues raised by the load-bearing structure of the platforms covering, amplified by the difficult foundation[11] conditions, have claimed at first the intake of engineering professionals, the desire to meet the need of representing the Dutch nation was to prove decisive to assign the work to an architect, rather then to an engineer. No less important to note that the interior decorative elements and arrangements were made in close cooperation with artistic professionals (Langmead, 2001: 14-15)[24]. The complexity of such constructions, both in terms of needs which had to be fulfilled and means of execution, made the collaboration between several specialists inevitable.



a *b* *c* *d*

Figure 13. a - Anhalter Bahnhof, b - roof montage; c - Amsterdam Central Station, d - interior view.

5. Conclusions

The use of iron load-bearing structures or structural components in Neoclassical, Neogothic, or Eclectic architecture, do not automatically generate consequences at the level of architectural space or form. In most cases, such use may go unnoticed. Exposed or not, in such cases, iron load-bearing structures or structural components are obedient to formal decisions imposed by the established architectural language.

Noticeable changes at the level of architectural space and form were rather the consequence of cumulating factors, between whom some unprecedented requirements, that only unusual structural forms were able to meet, have been instrumental. In such cases, the architecture deliberately takes, not only the load-bearing system but also the formal typology that the use of the load-bearing system has generated. Embedding of such major parts in the established architectural language, while maintaining the overall coherence, has required a substantial effort to restructure the architectural concept. This has been done in two major ways: by accepting eclectic solutions in which the composition and the decoration have sought to control the mixture resulting from the use of the best of that what art and technology offered at the time (Eclecticism); by challenging, and critical use of, the principles that have generated the accepted architectural language (Neoclassical and Neogothic structural rationalism). We can even assume that decisions on acquisition of new load-bearing systems were influenced by the ability of the architects to predict and control the outcome.

For structural form to reach a major influence on architectural form, including the level of decorative language, to allow for its expressive potential to be highlighted, structural development was not sufficient. It was necessary that architects find, on the principles that provided the basis of

[11] The building seats on three artificial islands in the river IJ, been founded on 26,000 wooden piles (Langmead, 2001: 14)[24]

established, accepted, architectural language, ways to organize, control and mediate the expression of the building as a whole. Thus, in the historicist architecture of the nineteenth century, the potential of the iron load-bearing structures has been exploited only to the extent that the architects have had the ability to foresee and control the enormous expressive power associated with them. This conclusions underline the fact that, although crucial in producing changes, the role of the iron load-bearing structure, was not to define formal characteristics of the historicist architecture. Its role was rather that of a catalyst for change, for opening or even 'deconstructing' the established architectural language.

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Vulnerability to progressive collapse of steel structures: GSA 2003 Guidelines

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Abstract

In this paper, the basic principles used for minimizing the potential for progressive collapse in the design of buildings are discussed. The main purpose of the GSA 2003 Guidelines is to reduce and to assess the potential for progressive collapse of the structures subjected to abnormal loads. The three basic approaches used in progressive collapse analysis are presented. Furthermore, this article discusses in detail the linear static analysis procedure for steel structures, according to GSA 2003 Guidelines: loading criteria, analysis cases, analysis criteria and acceptance criteria. To prevent progressive collapse, the seismic design of buildings should provide an adequate level of redundancy, continuity and ductility necessary for the structure to develop alternative load paths as a result of losing a member. In addition, this article presents results from papers where the analysis procedures for assessing the potential to progressive collapse for steel structures, following the GSA 2003 Guidelines, are presented.

Rezumat

În acest articol se discută principiile de bază utilizate pentru minimizarea potențialului la colaps progresiv în proiectarea clădirilor. Scopul principal al Ghidului GSA 2003 este de a reduce și evalua potențialul de colaps progresiv pentru structuri supuse unor incărcări accidentale. Sunt prezentate cele trei abordări de bază utilizate în analiza la colaps progresiv. Mai mult, acest articol discută în detaliu procedura de analiză statică liniară, aplicată structurilor metalice în conformitate cu prevederile Ghidului GSA 2003; de asemenea, sunt enumerate criteriile de încărcare, cazurile de analiză, criteriile de analiză și de acceptanță care sunt utilizate împreună cu procedura de analiză statică liniară. Pentru prevenirea colapsului progresiv, proiectarea seismică a clădirilor trebuie să asigure un nivel adecvat de redundanță, ductilitate și continuitate necesare pentru ca structura să dezvolte căi alternative ca urmare a pierderii unui element vertical de rezistență. În plus, acest articol prezintă rezultate din articole de specialitate în care se face analiza potențialului de colaps progresiv la structuri metalice, utilizând prevederile Ghidului GSA 2003.

Keywords: progressive collapse, seismic design, GSA 2003 Guidelines, abnormal loads, steel structures, alternative load path.

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1. Introduction

The notion of progressive collapse signifies o situation in which a local structural element failure appears that leads to the collapse of adjoining elements that at the end are amplifying gradually and thus the failure becomes progressive [1].

As a result of partial damage by collapse of Ronan Point building in England, in 1968, caused by an explosion at an upper level of the building, the issue of integrating measures of preventions from progressive collapse in structural design was debated. England was the first country to implement explicitly progressive collapse in structural design.

English Standards employ, for resisting progressive collapse design, the following approaches:

i. Tie Forces Method, an indirect approach which improves continuity, ductility and structural redundancy by the request to "tie" the structure in case of abnormal loads;
ii. Alternative Path Method, a direct approach that require that the structure is capable to bridge over the removed member as a result of abnormal loads;
iii. Enhanced Local Resistance, a direct method which assumes increasing the resistance of a key member, usually a column in the case of frame structure, for which the structure is not capable to bridge over the failed element.

Another important event, the bomb terrorist attack at Murrah Federal Building in Oklahoma City, in 1995, determined the United States to establish a set of design guidelines on progressive collapse such as:

i. "Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Project" published by the U.S. General Service Administration (GSA), in 2000;
ii. The U.S. General Service Administration (GSA) published an improved version, in 2003 [1]; this version has a separate chapter dedicated to steel structures;
iii. "Design of buildings to resist progressive collapse" published by the U.S. Department of Defense (DoD) in 2005 [2].
iv. The U.S. Department of Defense published an improved version, in 2009 [3];

The methodology presented in DoD 2005 Guidelines [2] and DoD 2009 Guidelines [3] follows the principle of "Load and Resistance Factor Design" (LRFD) from ASCE 7-02 [4] by introducing the load factor combinations and resistance factors to define design strengths. In addition, DoD 2009 Guidelines [3] has also implemented the "m-factor approach" from FEMA 274 [5] and FEMA 356 [6] which is similar to the approach discussed in GSA 2003 Guidelines [1].

The GSA and DoD Guidelines recommend to use "Alternative Path Method". Problems and commentaries related to the provisions of DoD 2005 Guidelines [2] and DoD 2009 Guidelines [3] will be disscussed in future studies. This article only refers to GSA 2003 Guidelines [1].

2. Progressive Collapse Analysis according to GSA 2003 Guidelines

2.1 General issues

The general purpose of these guidelines is to reduce the potential to progressive collapse of new and existing buildings. A steel structure will develop a low potential to progressive collapse if it meets three important features: to have sufficient redundancy, ductility and continuity. The GSA 2003 Guidelines [1] recommends the use of the "Alternative Path Method" which implies that the structure develops alternative ways of transmitting loads because of losing vertical support. As a result, the main feature of horizontal members is to be capable to resist the double spam condition; this is assured by [1]:

i. discrete beam-to-beam continuity: is considered "fundamental" to mitigating progressive collapse in steel frame structures; this clearly defined beam-to-beam continuity link across a column is capable of independently redistributing gravity loads for a multiple-span condition;

ii. connection resilience: is considered “essential” to mitigating progressive collapse in steel frame structures and provides a connection geometry that exhibits the physical attributes needed to mitigate the effects of instantaneous column loss;
iii. connection redundancy: is considered “essential” to mitigating progressive collapse in steel frame structures; therefore, it is important to select the proper beam-to-column-to-beam connection that provides positive, multiple and clearly defined beam-to-beam load paths;
iv. connection rotational capacity: the ability of a beam in a steel frame structure to develop a double span condition, created by a missing column scenario, is considered “fundamental” in mitigating progressive collapse;

The incorporation of these four characteristics will provide for a much more robust steel frame structure and increase the probability of achieving a low potential for progressive collapse when performing the linear static analysis procedure [1].

In the analysis of structures to progressive collapse, the following procedures are recommended:

i. Linear static or dynamic analysis which is used in the case of structures that are nominally 10 stories above grade or less;
ii. Nonlinear static or dynamic analysis, which takes into consideration the material and geometric nonlinearity, and it is used for structures that have over 10 stories, with typical or atypical structural configurations;

2.2 Linear static analysis

For the linear static procedure, GSA 2003 Guidelines [1] specify:

i. Specific loading criteria;
ii. A set of analysis cases in the so called “missing column” scenarios;
iii. Analysis criteria for the maximum allowable extents of collapse;
iv. Acceptance criteria for the results furnished by the linear static analysis;

All this criteria will be described in detail in the following sections.

2.2.1 Specific loading criteria

For static analysis a gravity load to the entire structure, according to Eq. (1) will be applied.

$$Load=2\ (DL+0.25LL) \tag{1}$$

where:

DL=dead load;
LL=live load;

In the GSA criteria, live load is reduced to 25% of the full design live load, admitting that the entire LL value is less probable. At the same time, by multiplying the load combination by a factor of two, the GSA 2003 Guidelines [1] take into account – in a simplified approach – the dynamic effect that occurs when a vertical support is instantaneously removed from the structure, and demands (Q_{UD}) in structural components are determined in terms of moments, axial forces, shear forces, etc [7].

2.2.2 Analysis cases

In the assessment methodology for the potential to progressive collapse according to GSA 2003 Guidelines [1] and DoD 2009 Guidelines [2], engineers should consider the loss of portions of the structure using different "missing column" or "missing beam" scenarios (see Fig. 1): an exterior column near to the middle of the short side (case C1), an exterior column near to the middle of the long side (case C2), a column located at the corner of the building (case C3) and an interior column (case C4) [7].

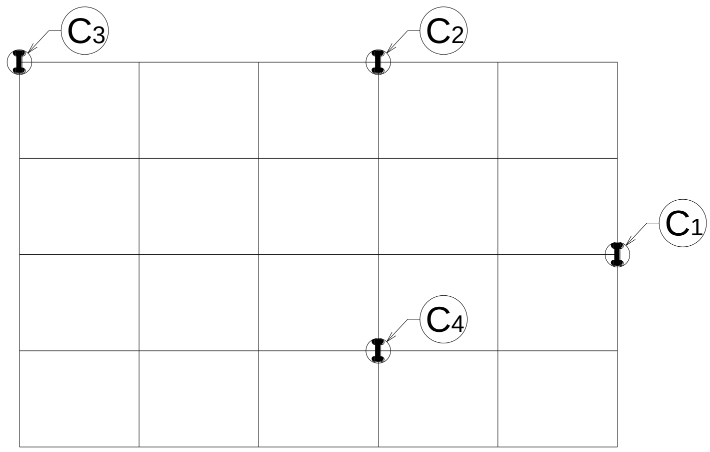

Figure 1. Analysis cases for the instantaneous loss of columns [1].

2.2.3 Analysis criteria

Structural collapse resulting from the instantaneous removal of a primary vertical support shall be limited. Typically, the allowable collapse area for a building will be based on the structural bay size [1]. However, there is the possibility that the bays between beams to be too large, as a consequence, the collapse region shall be limited to a reasonable area, given in Table 1.

Table 1: Maximum allowable collapse area

The lowest value for each case, between:		
1 st case: exterior removed column	Bays associated to removed column	*1800 feet²≈548m²*
2nd case: interior removed column	Bays associated to removed column	*3600feet²≈1097m²*

2.2.4 Acceptance criteria

Working with the results given by the linear elastic analysis (moment, shear, axial force), engineers shall identify the magnitude and distribution of potential areas of inelastic demands and thus, they will quantify the potential collapse areas [8]. The magnitude and distribution of these demands are indicated by DCR values (**D**emand-**C**apacity-**R**atios). This approach is also presented in guidelines such as: FEMA 274 [5] and FEMA 356 [6].

As a consequence, the DCR value for each component or connection is calculated as follows:

$$DCR = Q_{UD}/Q_{CE} \tag{2}$$

where:

Q_{UD} = acting force determined in member or connection (moment, axial force, shear or combined forces);

Q_{CE} = expected ultimate un-factored capacity of the member or connection in terms of moment, axial force, shear or combined forces;

The structural elements and connections with DCR values exceeding the allowable values given in Tab. 5.1 from the GSA 2003 Guidelines [1], are considered severly damaged or collapsed. In Tab.1 are summary presented the allowable DCR values depending on the section characteristics, material type and connection type.

Table 2: Acceptance criteria for linear static procedure [1]

Nr. crt.	Component/Connections types	DCR values
1.	Beams	2 → 3
2.	Columns	1 → 2
3.	Fully Restrained Moment Connections	2
4.	Partially Restrained Moment Connections	1.5 → 3

In the case of structures with atypical configuration, the allowed DCR values according to GSA 2003 Guidelines [1], will be reduced by ¼ of the DCR values presented in Table 2.

2.2.5 *Step-by-step linear static analysis procedure*

The linear static analysis procedure from the GSA 2003 Guidelines [1] implies the following steps:

1) Remove the column, load the structure according to Eq.1 and conduct a linear static analysis of the model.
2) Calculate DCR values for each component and connection; if the DCR value for shear is exceeded, the component is considered failed. In addition, if the flexural DCR value for both ends of the element are excedeed, creating a three hinge failure mechanism, the element is also considered failed. The elements considered failed are removed from the structural model.
3) For the elements that have DCR that exceed the allowed limit, a plastic hinge is introduced at the end of the element to release the moment; the hinge should be located at the center of flexural yielding but no more than half the depth of the member measured from the face of the intersecting member with the vertical one.

4) Apply equal and opposite moments on both sides of the hinge numerically equal with the expected flexural strength as represented in Fig. 2. The direction of the moments should be consistent with direction of the moments in the linear static analysis.
5) The analysis is run again and steps 1 to 3 are repeated. If the moments have been distributed to the entire structure and DCR values exceed the allowed limit, the structure is considered to have a high potential for progressive collapse.

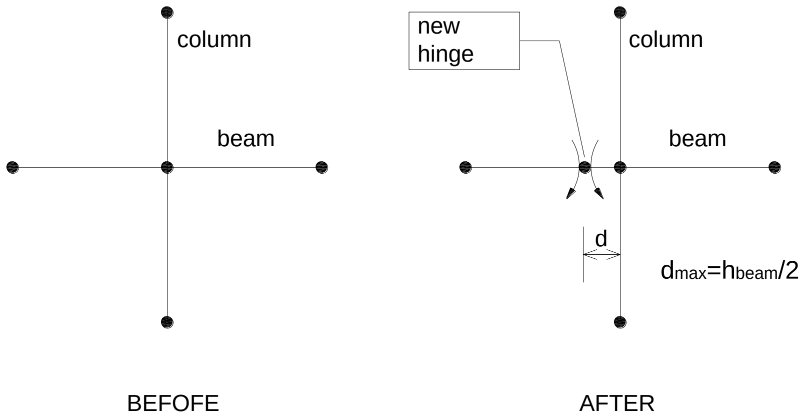

Figure 2. Insertion of plastic hinge and bending moments according to GSA 2003 Guidelines [1].

3. Frame structures: results and commentaries

Recently, Baldridge & Humay (2005) [9] have shown that a 12 story frame structure, seismically designed for a moderate (Zone 2B) or high seismic risk zone (Zone 4), does not experience progressive collapse when subjected to the “removal” of an external column. The study presents the methodology developed by the U.S. GSA (2003) for assessing the vulnerability of existing RC frame structures, as well as, results that confirm the inherent capacity of such structures seismically designed, to resist progressive collapse [8].

Bilow & Kamara (2003) [10] examine the application of progressive collapse analysis and designed guidelines included in the GSA 2003 Guidelines [1]. The building used in the study is a 12 stories RC frame structure designed for three different seismic design categories (A, C and D). The study shows that the columns, from the RC frame structure, in each of the three seismic zones do not require additional reinforcement to prevent progressive collapse. In addition, the beams designed to satisfy the strength requirements for seismic designed category D (high seismic zone), have sufficient strength to resist progressive collapse, unlike the beams from the others categories (A, C) which require additional reinforcement [10].

J. Kim & T. Kim (2008) [11] investigate the capacity of steel frames structures to resist progressive collapse using linear static, linear dynamic and nonlinear dynamic analysis procedures recommended in the GSA and DoD Guidelines. The structure used in the analysis has three different types of heights: three, six and fifteen stories. The study shows the differences between the use of linear static analysis procedure from GSA 2003 Guidelines [1] and DoD 2005 Guidelines [2] in the assessment of the potential to progressive collapse in buildings. In the DoD 2005 Guidelines [2] wind load is included in the load combination and the load factor for gravity load is larger than the load factor from GSA 2003 Guidelines [1]. Furthermore, the DoD 2005 Guidelines [2] recommend more rigorous criteria for assessing the potential to progressive collapse in buildings, than the GSA 2003 Guidelines [1]. The study also shows the differences between the linear static and dynamic analysis procedure from GSA 2003 and DoD 2009 Guidelines. In Fig.3 it can be observed that less hinges were formed as a result of dynamic analysis and the DCR values obtained from dynamic analysis were also less than those computed by static analysis [11].

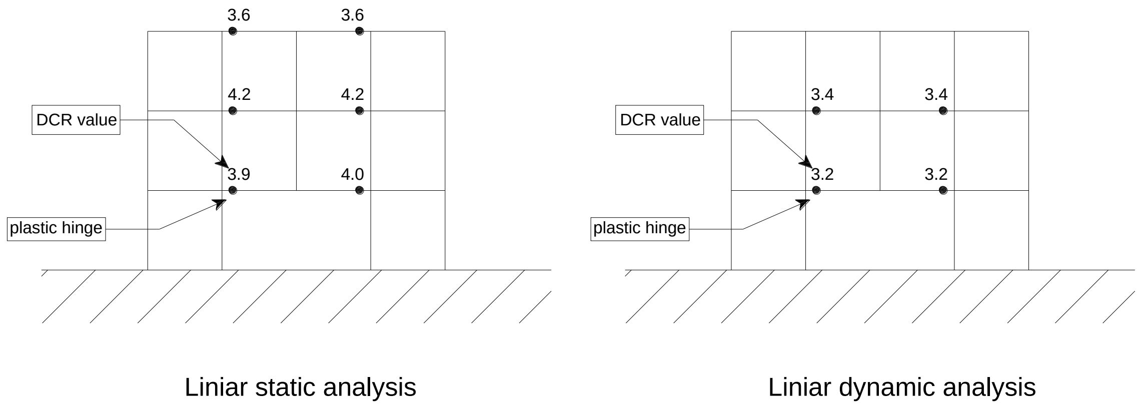

Figure 3. Comparison of plastic hinges locations and DCR values for linear static versus dynamic analysis (Kim J. & all) [11].

4. Conclusions

1.This study is in line with the trends of the specialized reference literature that aims at assessing the vulnerability of the existing structures subjected to abnormal or catastrophic loads produced by natural hazard (e.g. earthquakes) or by man-made hazards (terrorist attacks, impact by vehicles, bomb blast, etc).
2. Irrespective of the nature of the threat, it is not rational to examine all specific potential sources of collapse initialization. For this reason, the removal of a column – in the GSA 2003 and in DoD 2005 and 2009 Guidelines – is regarded as a “load initiator”, in order to examine the redundancy and the resilience of the structure.
3. Practically, due to the economic constraints, it is impossible to design an overall structure and each structural member individually so as to resist to abnormal loads or to prevent collapse initiation from a specific cause. It is more important to stop or to limit the progression of the collapse and to reduce the extent of the damage and this should be the design philosophy assumed by engineers.
4. Many design codes (British Standards, ACI 318, EC-2, National Building Code of Canada, Swedish Design Regulation, P100-92, P100-1/2006) require an adequate level of continuity, redundancy and ductility for the selected structural system; these requirements are found in seismic design, too [8].
5. Theoretical studies (Baldridge & Humay), (Bilow & Kamara), (Ioani & all) [12], have shown that medium-rise building having RC framed structures and seismically designed for zone of moderate or high seismic risk do not experience progressive collapse when subjected to the removal of an exterior or interior column.
6. Similar analyses to progressive collapse using different procedures (GSA and DoD Guidelines), have been made on steel frame structures by Kim [11]. It has been underlined that the steel moment frames designed for lateral and gravity loads are less vulnerable to progressive collapse than the similar frames designed only for gravity loads. In the same time, the potential to progressive collapse of steel structures decreased as the number of story increased (from 3 to 6 or to 15 stories).
7. The GSA 2003 Guidelines [1] offer a realistic approach and performance criteria to evaluate the potential to progressive collapse of frame structures [8], [9], [10] and [11]; the linear static procedure used in GSA 2003 Guidelines is theoretical simple and can be conducted without sophisticated nonlinear modeling, and leads to a more conservative decision than the nonlinear dynamic time-history method [11].
8. Based on authors’ expertise in the seismic design as well as in the analysis of RC framed structures to progressive collapse, a research program concerning the vulnerability to progressive

collapse of steel structures seismically designed started, and analyses of 6-story steel structure designed for Bucharest (a_g=0.24g) are in progress; the risk to progressive collapse is evaluated following the linear static procedure from the GSA 2003 and DoD 2009 Guidelines, and the results will be discussed and compared.

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Comparative Analysis Concerning the Load Capacity of a Railway Bridge. Romanian Norms - Eurocode

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Abstract

This paper presents a comparative analysis concerning the load bearing capacity of the bridge main girders evaluated in accordance with the Romanian norms and respectively with the European norms (Eurocodes).The analyzed girders are part of a 15 m span railway steel bridge.

Rezumat

În lucrare se analizează comparativ starea de eforturi din încovoiere pentru un tablier de cale ferată simplă cu deschiderea de 15 m, pe grinzi principale metalice fără antretoaze, analiză efectuată conform normelor române şi conform euronorme. Analiza rezultatelor obţinute pe baza evaluării acţiunilor şi a verificării la încovoiere, în conformitate cu normele române şi respectiv în conformitate cu normele europene, se face în raport cu gradul de solicitare a grinzilor principale faţă de limita maximă admisă. În exemplul de calcul analizat, rezultatele obţinute prin cele două metode de evaluare a capacităţii portante la încovoiere a grinzilor tablierului sunt apropiate, diferenţele fiind cuprinse în intervalul 3 - 4%.

Keywords: RAILWAY BRIDGE, LOAD CAPACITY, ROMANIAN NORMS, EUROCODE

1. Introduction

Until the adoption of the European norms the allowable resistances method using T 8.5 convoy, Figure 1, were used in our country for the design of steel bridges.

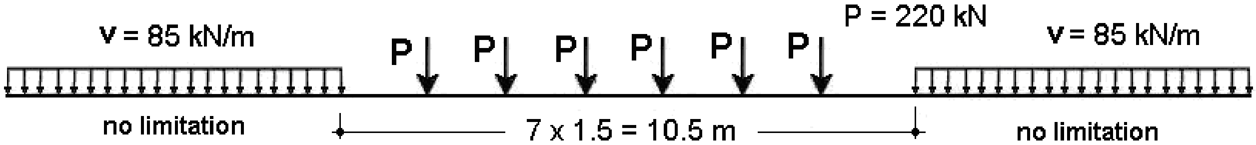

Figure 1. T8,5

According to EN 1991-2 “Actions on structures - Part 2: Traffic loads on bridges”, Section 6, the

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rail traffic actions are defined by means of load models.
Five models of railway loading are given:

- **LM 71** – represents the static effect of vertical loading due to normal rail traffic, Figure 2.

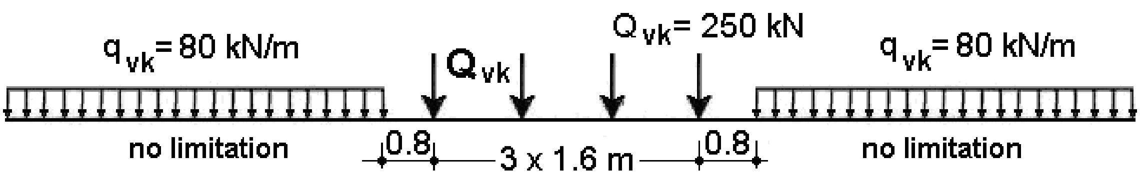


Figure 2. Load Model 71 and characteristic values for vertical loads

- **SW/0 and SW/2**

Load Model SW/0 represents the static effect of vertical loading due to normal rail traffic on continuous beams.
Load Model SW/2 represents the static effect of vertical loading due to heavy rail traffic.
The load arrangement shall be considered as shown in Figure 3, with the characteristic values of the vertical loads according to Table 1.

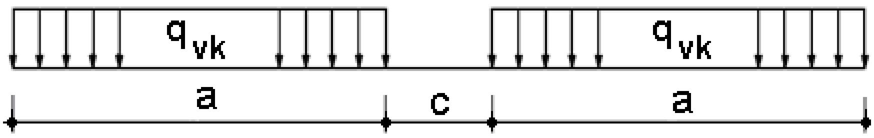


Figure 3. Load Models SW/0 and SW/2

Table 1

Load Model	q_{vk} [kN/m]	a [m]	c [m]
SW/0	133	15,0	5,3
SW/2	150	25,0	7,0

- **Load Model " unloaded train"**

For certain specific verifications (see EN 1990, A.2) the load model which consists of a vertical uniformly distributed load with a characteristic value of 10.0 kN/m is used.

- **Eccentricity of vertical loads**

The effect of lateral displacement of vertical loads shall be considered by taking the ratio of wheel loads on all axles as up to 1.25:1.00 on any track.

This paper presents a comparative analysis regarding the bending verification of a railway bridge using Romanian norms and Euro norms.

2. Comparative analysis

The state of bending stresses of a steel railway bridge according to Romanian norms and to euronorme is comparatively analysed knowing the following design data:

- bridge span; L=15.00 m;
- structural steel: OL 52.4k (S 355. K2.M);
- geometrical scheme and transversal section – see Figure 4

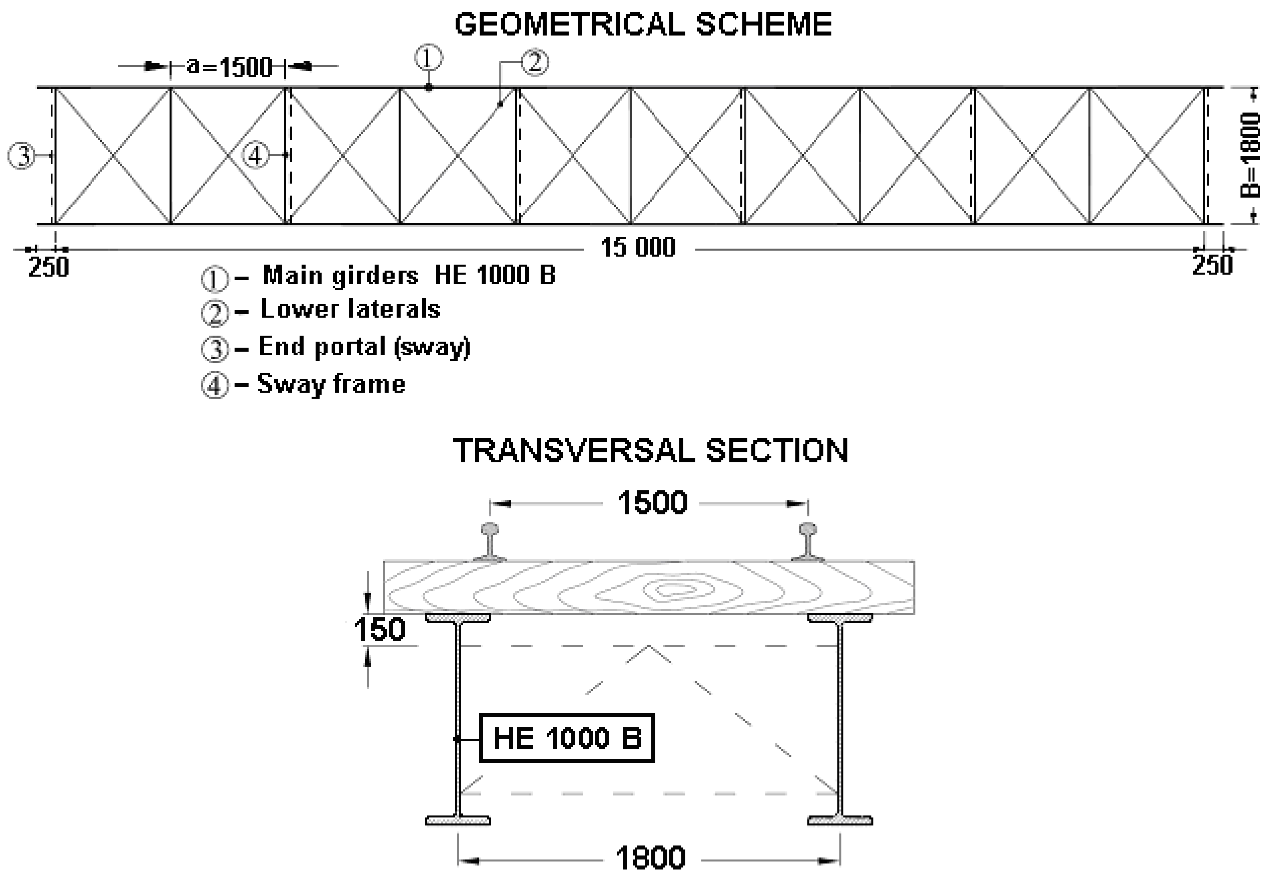


Figure 4

2.1 Verification according to SR 1911-9

The main girders are checked for two combinations of actions (groups of loads):
Combination of actions (group of loads) I:

- permanent actions;
- vertical convoy actions.

Combination of actions (group of loads) II:

- permanent actions;
- vertical convoy actions;
- nosing force;
- wind action.

Actions and bending moments on main girders

Permanent actions

According to *Norms for design of railway bridges. Actions/2004* (reviewing of STAS 1489-78) the following permanent loadings are evaluated:

Track weight: $g_1 = 9.00$ kN/m

Superstructure weight: $g_2 = k_8(0.44 \cdot L + 6.5) = 0.85\,(0.44 \cdot 15 + 6.5) = 11.14\ \text{kN/m}$

It results: $g_{perm} = g_1 + g_2 = 20.14\ \text{kN/m}$

For one main girder: $g = \dfrac{g_{perm}}{2} = \dfrac{20.14}{2} = 10.07\ \text{kN/m}$

The bending moment produced by permanent actions will be:

$$M_{g.max} = \frac{g \cdot L^2}{8} = \frac{10.07 \cdot 15^2}{8} = 283.22\ \text{kNm}$$

Actions and moments given by convoy T 8.5 and wind pressure

For simple supported girders the bending moment produced by convoy t 8.5 is given by the relation:

$$M_{max.max} = 10.65\,L^2 + 106.8\,L - 320\ \ [\text{kNm}]$$

For a single main girder the maximum bending moment will be:

$$M_{T8.5.max} = \frac{1}{2}(10.65 \cdot 15^2 + 106.8 \cdot 15 - 320) = 1839.125\ \text{kNm}$$

Dynamic factor ψ :

$$\psi = 1.10 + \frac{17}{35+L} = 1.10 = \frac{17}{35+15} = 1.44 \text{ - for welded rails}$$

The maximum bending moment for the group of loads I results:

$$M_{max} = M_{g.max} + \psi \cdot M_{T8.5.max} = 283.22 + 1.44 \cdot 1839.125 \approx 2932\ \text{kNm}$$

For the group of loads II will be added the action of nosing force and wind pressure.

Direct action of nosing force

The nosing force of a value S=60 kN produces a local bending of the upper flange of the main girder and also a compression force as is shown in Figure 5.

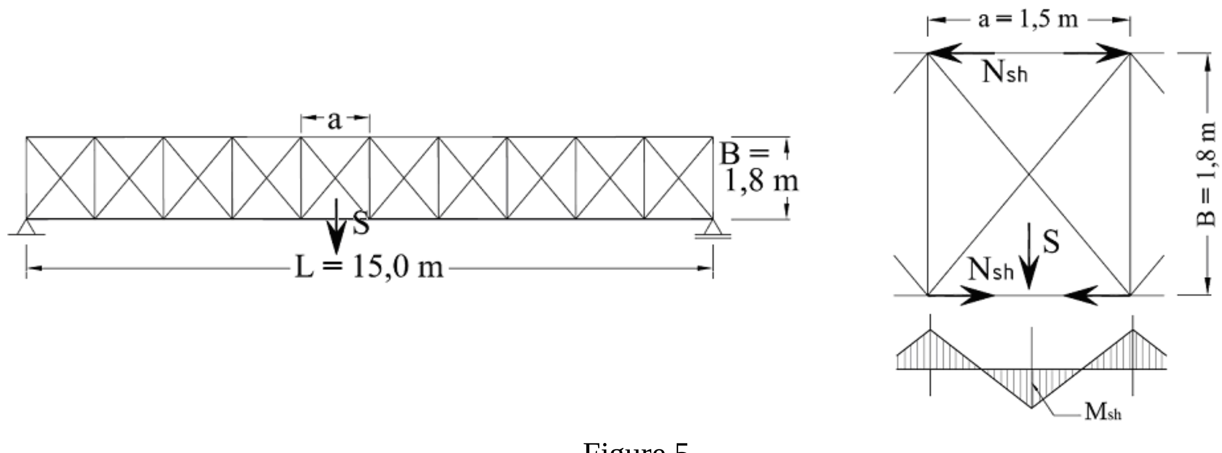


Figure 5

Bending moment can be taken:

$$M_{Sh} = \frac{1}{4} M_{Oh} = \frac{1}{4}\left(\frac{S \cdot a}{4}\right) = \frac{1}{4}\,\frac{60 \cdot 1.5}{4} = 5.625\ \text{kNm}$$

Axial compression force:

$$N_{Sh} = \frac{M_{Sh}^{(L)}}{B} = \frac{1}{B}\left(\frac{S \cdot L}{4}\right) = \frac{1}{1.8}\left(\frac{60 \cdot 15}{4}\right) = 125\ \text{kN}$$

Direct action of wind

Similar with the direct action of the nosing force, the wind pressure of value $p_w = 150\ daN/m^2$ (1.5 kN/m^2) will produce local bending and compression of the upper flange, where:

- bending moment: $M_{wh} = \dfrac{w \cdot a^2}{20} = \dfrac{7.35 \cdot 1.5^2}{20} = 0.827\ kNm$

with: $w = (h_{conv} + h_{cale} + h_{gr.pr.}) \cdot p_w = (3.5 + 0.4 + 1.0) \cdot 1.5\ kN/m^2 = 7.35\ kN/m$

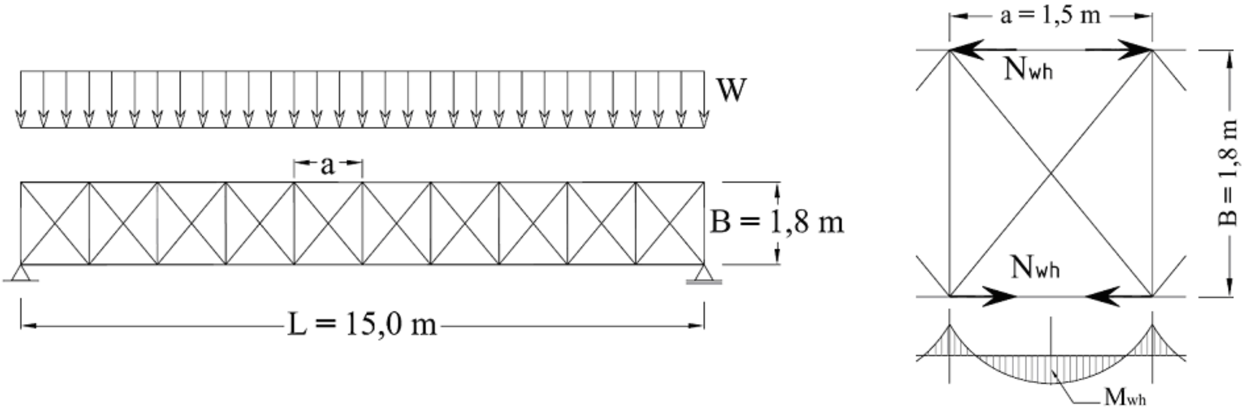


Figure 6

- axial compression: $N_{wh} = \dfrac{M_{wh}^{(L)}}{B} = \dfrac{1}{B}\left(\dfrac{w \cdot L^2}{8}\right) = \dfrac{1}{1.8}\left(\dfrac{7.35 \cdot 15^2}{8}\right) = 114.84\ kN$

By summing the nosing force and wind actions will result:

- bending moment: $M_h = M_{Sh} + M_{wh} = 5.625 + 0.827 = 6.452\ kNm$
- axial compression: $N_h = N_{Sh} + N_{wh} = 125 + 114.84 = 239.84\ kN$

Indirect action of nosing force and wind pressure

Because the nosing force and wind resultant are acting with an eccentricity relatively to horizontal lower laterals (Figure 7), a supplementary loading of the main girders will take place.

Indirect action of nosing force

The nosing force acts with an eccentricity: $d_S = 0.55\ m$.

It results:

$$S_{ind} = \frac{S \cdot d_S}{B} = \frac{60 \cdot 0.55}{1.8} = 18.33\ kN$$

$$M_{S.ind} = \frac{S_{ind} \cdot L}{4} = \frac{18.33 \cdot 15}{4} = 68.74\ kNm$$

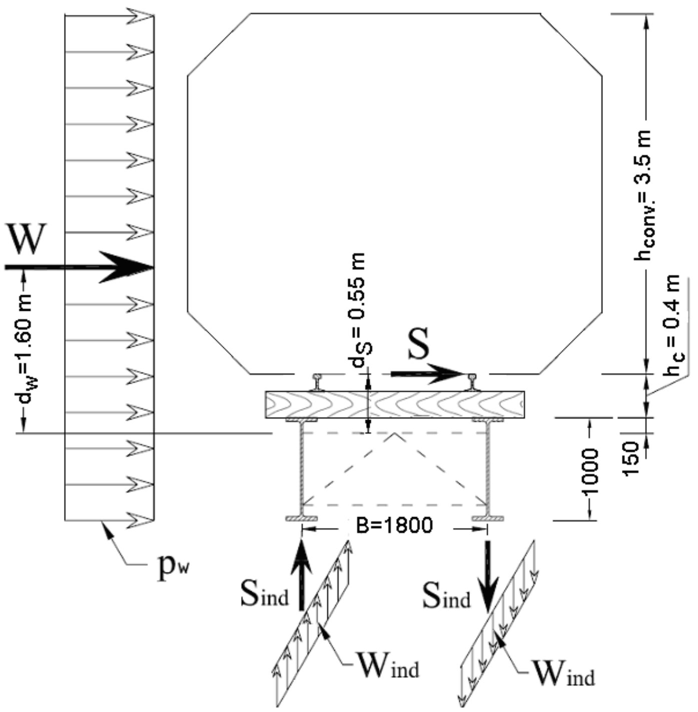


Figure 7

Indirect action of wind pressure

The wind pressure resultant acts with an eccentricity: $d_w = 1.6$ m
It results:

$$w_{ind} = \frac{w \cdot d_w}{B} = \frac{7.35 \cdot 1.6}{1.8} = 6.53 \text{ kN/m}$$

$$M_{w.ind} = \frac{w_{ind} \cdot L^2}{8} = \frac{6.53 \cdot 15^2}{8} = 183.66 \text{ kNm}$$

The total supplementary bending moment given by the indirect action of the nosing force and wind pressure will be:

$$M_{ind} = M_{S.ind} + M_{w.ind} = 68.74 + 183.66 = 252.4 \text{ kNm}$$

Verification of girder to bending

Group of loads I

It is checked the condition:

$$\sigma = \frac{M_{max}}{W_{el.y}} \le \sigma_a^I$$

It is obtained:

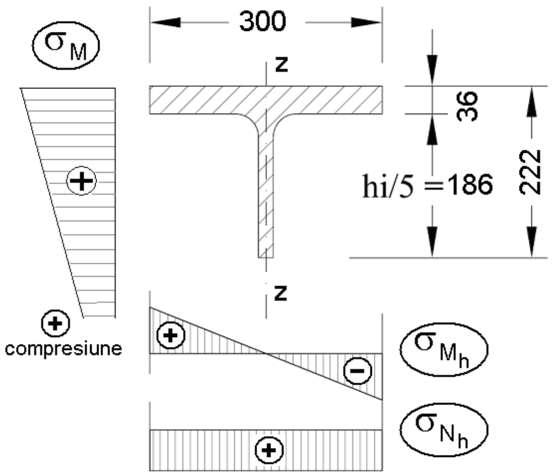
$$\sigma = \frac{2\,932 \cdot 10^4}{12\,890} = 2275 \text{ daN/cm}^2 < \sigma_a^I = 2400 \text{ daN/cm}^2$$

Group of loads II

It is checked the condition:

$$\sigma = \frac{M_{max} + M_{ind}}{W_{el.y}} + \frac{M_h}{W_{z.t}} + \frac{N_h}{\varphi_{z.t} \cdot A_t} \le \alpha \cdot \sigma_a^{II}$$

where (Figure. 8):



$$I_{z.t} = 8\,124\ \text{cm}^4$$

$$A_t = 147.18\ \text{cm}^2$$

$$i_{z.t} = \sqrt{\frac{I_{z.t}}{A_t}} = \sqrt{\frac{8124}{147.18}} = 7.43\ \text{cm}$$

$$\lambda_{z.t} = a / i_{z.t} = 150 / 7.43 = 20.18 \Rightarrow \varphi_{z.t} = 0.87$$

$$W_{z.t} = I_{z.t} /(b/2) = 8124/15 = 541.6\ \text{cm}^3$$

Figure 8

It results:

$$\sigma = 2\,776\ \text{daN}/\text{cm}^2 < 1.1 \cdot 2700 = 2\,970\ \text{daN}/\text{cm}^2$$

2.1 Verification according to Eurocodes

Actions and bending moments of main girders evaluation

Combination of actions

The following parameters in the ultimate limit states design shall be taken into account:

- γ - is a partial factor for the action which takes account of the possibility of unfavourable deviations of the action values from the representative values;
- ψ - is a action combination factor which can be as follows:
  - ψ_0 - combination factor for variable actions;
  - ψ_1 - combination factor for frequent values of the variable actions;
- ψ_2 - combination factor for quasi-permanent values of the variable actions.

Combination of actions is made with the relation:

$$\gamma_G \cdot G + \gamma_{Q.1} \cdot Q_{k.1} + \sum \gamma_{Q.i} \cdot \psi_i \cdot Q_{k.i}$$

In case of the railway bridges the above factors have the following values:

- $\gamma_G = 1.35$; $\gamma_{Q.i} = 1.45$; $\psi_0 = \psi_1 = 0.8$; $\psi_2 = 0$

Permanent actions:

g=10.07 kN/m; $M_{g.max} = 283.22$ kNm

The bending moment for a main girder:

$$M_{Eg} = \gamma_G \cdot M_{g.max} = 1.35 \cdot 283.22 = 382.35 \text{ kNm}$$

Convoy action:

The superstructure will be verified for Load Model 71 and for simplicity the equivalent loading will be used which for a span L=15 m has a value: $q_m = 132.5 \text{ kN/m}$

Dynamic factor:

The dynamic factor Φ is as follows:

- for a good maintenance of the railway: $\Phi_2 = \dfrac{1.44}{\sqrt{L_\Phi} - 0.2} + 0.82 \qquad 1.00 \le \Phi_2 \le 1.67$
- for a standard maintenance of the railway: $\Phi_3 = \dfrac{2.16}{\sqrt{L_\Phi} - 0.2} + 0.73 \qquad 1.00 \le \Phi_3 \le 2.00$

L_Φ - determinant length, defined in Table 6.2 of EC1 - Part 2. For simple supported girder: $L_\Phi = L$.

In this case: $\Phi = \Phi_3 = 1.32$ - for $L_\Phi = 15$ m

The bending moment given by LM 71 actions:

$$M_{EP} = \gamma_Q \cdot \Phi_3 \cdot M_{LM71} = 1.45 \cdot 1.32 \cdot 1\,863.28 = 3\,566.32 \text{ kNm}$$

where: $M_{LM71} = \dfrac{1}{2}\dfrac{q_m \cdot L^2}{8} = \dfrac{1}{2}\dfrac{132.5 \cdot 15^2}{8} = 1\,863.28 \text{ kNm}$

The design bending moment will be:

$$M_{Ed} = M_{Eg} + M_{EP} = 382.35 + 3\,566.32 = 3949 \text{ kNm}$$

Nosing force: Q_{Sk}=100 [kN] (without dynamic factor Φ)

The bending moment produced by nosing force:

$$M_{ES.h} = \frac{1}{4}\frac{\gamma_{Sk} \cdot Q_{Sk} \cdot a}{4} = \frac{1}{4}\frac{1.45 \cdot 100 \cdot 1.5}{4} = 13.59 \text{ kNm}$$

The axial force in the upper flange:

$$N_{ES.h} = \frac{1}{B}\frac{\gamma_{Sk} \cdot Q_{Sk} \cdot L}{4} = \frac{1}{1.8}\frac{1.45 \cdot 100 \cdot 15}{4} = 302 \text{ kNm}$$

Wind action

The wind action is evaluated in accordance with EC1-Part 1-4.

For a terrain category II, reference height z_e =10 m, wind velocity v_b=27 m/s, it is obtained:

- C_e=2.15
- $q_p = 2.15\dfrac{1.25}{2}27^2 = 980 \text{ N/m}^2 = 98 \text{ daN/m}^2$

The wind force: $F_{w.x} = q_p(z_e) \cdot C_{f.x} \cdot A_{ref.x}$

$C_{f.x}$ – coefficient - Figure 8.3 of EC1 - Part 1-4.

In this case: $b / d_{tot} \approx$ =1 $\Rightarrow C_{f.x} = C_{fx.0} = 2.3$

It results: $F_{w.x} = 98 \cdot 2.3 \cdot A_{ref.x} \approx 225\,[\text{daN/m}^2] \cdot A_{ref.x}$

The bending moment produced by the wind action is:

$$M_{Ew.h} = \frac{1}{20}\gamma_w \cdot p_w \cdot a^2 = \frac{1}{20}1.5 \cdot (2.25 \cdot 4.9) \cdot 1.5^2 = 1.86\ kN \cdot m$$

The bending moment produced by horizontally forces ψ will be:

$$M_{Ed.h} = \psi_0 \cdot (M_{ES.h} + M_{Ew.h}) = 0.80 \cdot (13.59 + 1.86) = 12.36\ kN \cdot m$$

The axial compression force is:

$$N_{Ed.h} = \psi_0 \cdot (N_{ES.h} + N_{Ew.h}) = 0.80 \cdot (302 + 258.4) = 448.32\ kN$$

Indirect action of nosing force and wind pressure

Indirect action of nosing force

The nosing force acts with an eccentricity: $d_S = 0.55\ m$.

It results:

$$Q_{S.ind} = \gamma_{Sk} \frac{Q_{Sk} \cdot d_S}{B} = 1.45\frac{100 \cdot 0.55}{1.8} = 44.3\ kN, \qquad M_{ES.ind} = \frac{Q_{S.ind} \cdot L}{4} = \frac{44.3 \cdot 15}{4} = 166.125\ kNm$$

Indirect action of wind pressure

The wind pressure resultant acts with an eccentricity: $d_w = 1.6\ m$

It results: $w_{ind} = \gamma_w \dfrac{p_w \cdot d_w}{B} = 1.5\dfrac{(2.25 \cdot 4.9) \cdot 1.6}{1.8} = 14.7\ kN/m$

$$M_{Ew.ind} = \frac{w_{ind} \cdot L^2}{8} = \frac{14.7 \cdot 15^2}{8} = 413.44\ kNm$$

The total supplementary bending moment given by the indirect action of the nosing force and wind pressure will be:

$$M_{Ed.ind} = 0.8\,(M_{ES.ind} + M_{Ew.ind}) = 0.8\,(166.125 + 413.44) = 463.65\ \ kNm$$

Verification of girder to bending

Cross section Class

Compression flange:

$$\frac{c}{t} = \frac{(b-2r)/2}{t_f} = \frac{(300-2\cdot 30)/2}{36} = 3.3 < 9\cdot\varepsilon = 9\cdot 0.81 = 7.29 \Rightarrow \text{ flange Class 1}$$

Web:

$$\frac{c}{t} = \frac{h-2\,(t_f + r)}{t_w} = \frac{1000-2\,(36+30)}{19} = 45.68 < 72\cdot\varepsilon = 58.32 \Rightarrow \text{ web Class 1}$$

Cross section Class = max [flange Class=1; web Class=1] = 1

Design characteristics

HE 1000B:

$$W_{pl.y} = 14\,860\ cm^3, \quad A_t = 147.18\ cm^2, \ W^{t.sup}_{pl.z} = \frac{t_f \cdot b^2}{4} = \frac{3.6 \cdot 30^2}{4} = 810\ cm^3$$

Steel S 355.J2.M: $f_y = 345\ N/mm^2$ - for t >16 mm

Verification of girder to combined biaxial bending and compression

The verification of girder subjected to biaxial bending and compression of the upper flange is made according to SR EN 1993-1-1.§ 6.2.5 şi § 6.2.9:

$$\frac{M_{Ed.tot}}{M_{c.Rd}} + \frac{M_{Ed.h}}{M_{N.Rd.h}} \leq 1$$

where:

$$M_{Ed.tot} = M_{Ed} + M_{Ed.ind} = 4.41 \cdot 10^{7}\,\mathrm{daN \cdot cm}, \quad M_{Ed.h} = 1.236 \cdot 10^{5}\,\mathrm{daN \cdot cm}$$

$$M_{c.Rd} = M_{pl.Rd} = \frac{W_{pl.y} \cdot f_y}{\gamma_{M0}} = 5.126 \cdot 10^{7}\,\mathrm{daN \cdot cm}$$

$$M_{N.rd.h} = M_{pl.Rd.h}\left[1 - \left(\frac{N_{Ed.h}}{N_{pl.Rd.h}}\right)^2\right] = W_{pl.z}^{t.sup}\,\frac{f_y}{\gamma_{M0}}\left[1 - \left(\frac{N_{Ed.h}}{A_t f_y / \gamma_{M0}}\right)^2\right] = 2.77 \cdot 10^{6}\,\mathrm{daNcm}$$

It is obtained:

$$\frac{4.41 \cdot 10^{7}}{5.126 \cdot 10^{7}} + \frac{1.236 \cdot 10^{5}}{2.77 \cdot 10^{6}} = 0.905 < 1$$

5. Results analysis and final remarks

According to Romanian norms the design of steel bridges is made using the available stress method, until the euro norms are used.

In Table 2 the obtained results using Romanian design methods and euro norms relative to actions evaluation and steel members design are synthetically presented.

Table 2. Superstructure grade of stressing

STAS 1911- 98		EURONORMS
Group of loads I	94.8 %	90.5 %
Group of loads II	93.5 %	

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The oscillation period for moored vessels in Constanţa port

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Abstract

The present paper brings to attention the importance of the lines used to anchor the vessels in harbours. For the moored vessels, oscillation is one of the most important parameters. The elastic behaviour of the cables, made of various fibers, is difficult to determine, because it depends on material, composition, loading history and environmental conditions. The natural oscillation period of the moored vessel depends on the vessels displacement, number, type and loading of the lines. Several cases were studied, in order to determine the influence of the variables on the oscillation and vibration modes of vessels and basins. The oscillation period of the moored vessel was determined with the module „Surging of a moored vessel" implemented in the demo version of CEDAS. The results refere to the amplification factor and the possibility of resonance of the natural oscillations period of vessels and basins.

Rezumat

Prezentul articol aduce în atentie importanţa parâmelor utilizate pentru acostarea navelor. Pentru navele acostate, oscilaţia reprezintă unul din cei mai importanţi parametri. Comportarea elastică a parâmelor , realizate din diverse fibre, este dificil de determinat, deoarece depinde de material, compoziţie, istoricul încărcărilor şi condiţii de mediu. Perioada proprie de oscilaţie a navelor acostate depinde de deplasamentul navelor, tipul, numărul şi solicitarea parâmelor. Au fost studiate mai mullte cazuri, în vederea determinării influenţei variabilelor asupra oscilaţiei şi a modurilor de vibraţie pentru nave şi petru bazinul portuar. Perioada de oscilaţie a navelor acostate s-a determinat cu ajutorul modulului „Surging of a moored vessel" implementat în versiunea demo a programului CEDAS. Rezultatele fac referinţă la factorul de amplificare şi la posibilitatea de intrare în rezonanţă a perioadelor proprii de oscilaţie ale navelor şi bazinelor portuare.

Keywords: natural period, oscillation, line, basin, amplification factor

1. **Oscillations of the ship at berth**

1.1. Theoretical aspects

For the moored vessels, oscillation is one of the most important parameters. According to EM 1110-2-1100 Part II, the movement of a vessel tied to dock can be described as the oscillation of a linear

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system with 1 gld. The reactions due to the change of position, speed or acceleration of the vessel compared to the steady state are supposed to be linear. The disruptive forces are due to the pushing force of the water flowing along the vessel. The absorption is considered small for the low-frequency movement of the moored vessel. In these conditions, the natural period can be obtained using the equation 1

$$T_n = 2\pi\sqrt{\frac{m_v}{kt_{ot}}} \tag{1}$$

knowing that

- m_v is the virtual mass of the vessel, equal with the sum of the vessel displacement and an additional mass, due to the inertial effects of the water moved by the vessel. The additional mass for an oscillating vessel is about 15% from the real mass.

$m_v = 1.15m$

- k_{tot} represents the effective elastic constant, inferred solely on the stiffness of stretched lines (k_n)

$$k_{tot} = \sum_n k_n \sin\theta_n \cos\varphi_n \tag{2}$$

θ_n – angle made by the line in horizontal plan with the perpendicular on the vessel;
φ_n – angle made by the line in vertical plan, between vessel and dock .

- $k_n = \dfrac{N_n}{\Delta l}$ individual stiffness of the stretched line; it is defined as the ratio of the axial stress of the line and elongation (in numerical simulations realized with the CEDAS the axial stress was considered to be equal to the traction in bollard, estimated in compliance with the recommendations from the literature, according to the vessel displacement).

From the equations 1 and 2, it comes out that the natural oscillation period of the moored vessel depends on the vessels displacement, number, type ad loading of the lines. During the loading-unloading process the vessel displacement changes, which leads to changes of the dynamic characteristics. A suitable ballast or an adjustment of the lines stress or number, can avoid the resonance with the basin oscillation period.

The elastic behaviour of the cables, made of various fibers, is difficult to determine, because it depends on material, composition, loading history and environmental conditions. Usually, the producers provide experimental curves, where the specific deformation, expressed as a percentage, is given according to the load (expressed as percentage from the capable axial stress).

Under heavy loads, repeated over time, the fibers in the lines may exceed the elasticity limit and might break.

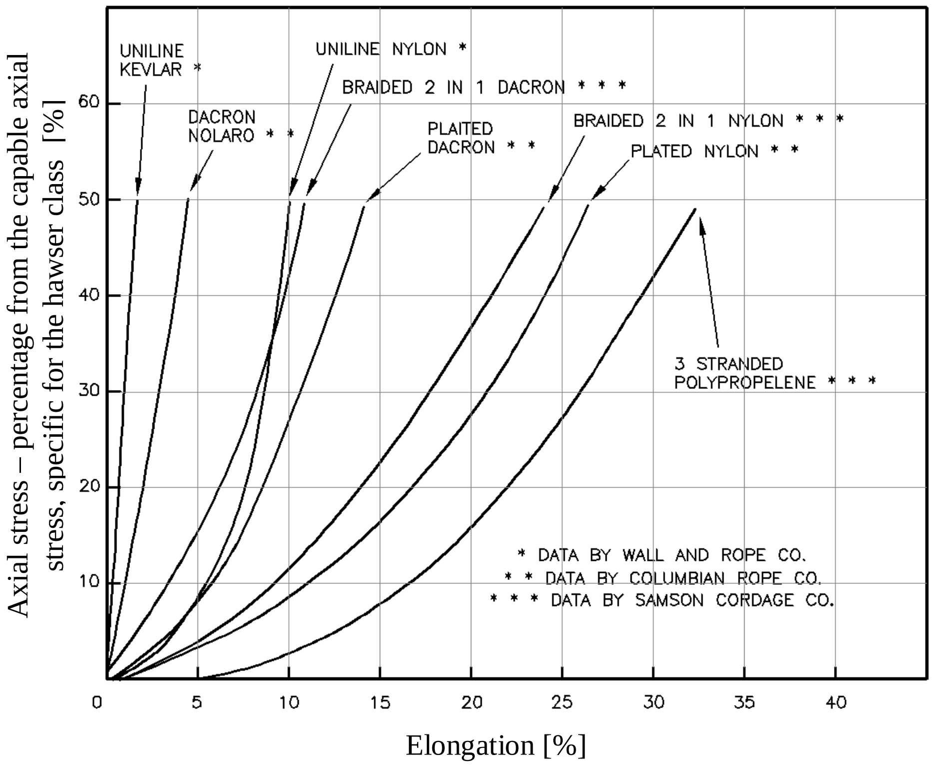

Figure 1. Elongation curves for lines made of various materials

1.2. Case Study

The oscillation period of the moored vessel was determined with the module „Surging of a moored vessel" implemented in the demo version of CEDAS.

Several study cases were taken into account. It was started with a typical case of a heavy vessel (50 000 tdw), knowing the axial stress in the line, the distance above dock, number, length and orientation of lines, elongation (according to the line type) – case 1.

Case nr.1

Bow or Head Line			
Distance above dock	7.000 m	Line length	30.500 m
Angle of line perpendicular to breast line	70.000 deg	Number of lines	2
Elongation	2.6535 m	Stiffness	67043.5 N/m
Vertical angle	13.2681 deg		
Stern Line			
Distance above dock	7.000 m	Line length	30.500 m
Angle of line perpendicular to breast line	70.000 deg	Number of lines	2
Elongation	2.6535 m	Stiffness	67043.5 N/m
Vertical angle	13.2681 deg		
Spring Line			
Distance above dock	5.000 m	Line length	45.700 m
Angle of line perpendicular to breast line	85.000 deg	Number of lines	2
Elongation	3.9759 m	Stiffness	44744.6 N/m
Vertical angle	8.81084 deg		
Virtual mass	6.4239e+007 kg	Load (% breaking strength)	33.8213%
Effective spring constant			
Forward	166686 N/m	Reverse	166686 N/m
Total	166686 N/m		
Natural period T_s	123.347 sec		

Case nr.2

Line tension	300000.000 N	Ship mass (m)	55860000.000 kg
Percent elongation	14.600	Breaking strength	526000.000 N/m
Bow or Head Line			
Distance above dock	7.000 m	Line length	30.500 m
Angle of line perpendicular to breast line	70.000 deg	Number of lines	2
Elongation	4.453 m	Stiffness	67370.3 N/m
Vertical angle	13.2681 deg		
Stern Line			
Distance above dock	4.500 m	Line length	30.500 m
Angle of line perpendicular to breast line	70.000 deg	Number of lines	2
Elongation	4.453 m	Stiffness	67370.3 N/m
Vertical angle	8.48445 deg		
Spring Line			
Distance above dock	4.500 m	Line length	45.700 m
Angle of line perpendicular to breast line	85.000 deg	Number of lines	2
Elongation	6.6722 m	Stiffness	44962.7 N/m
Vertical angle	5.65097 deg		
Virtual mass	6.4239e+007 kg	Load (% breaking strength)	57.0342%
Effective spring constant			
Forward	169803 N/m	Reverse	167809 N/m
Total	168806 N/m		
Natural period T_s	122.57 sec		

Compared to the reference case, there were modified various inputs. Mainly, the displacement of the vessel and the length of lines were changed. In another phase the axial stress in the line was also changed, correlated to the traction in bollard, as in table 1.

Table 1 Traction in bollard, according to the vessels displacement

	Displacement of the vessel (tdw)	**Traction in bollard (kN)**
1	<5000	50
2	5000-15000	100
3	15000-25000	150
4	25000-50000	200
5	<50000	250

The centralization of these studies is presented in table 2 and on this basis some conclusions can be drawn, applicable to vessels tied at docks.

Table 2 Centralized conclusions from the study of 12 cases

	case1	case2	case3	case4	case4a	case4b	case5	case6	case7	case8	case8a	case9
Displacement of vessel (tdw)	**50000**	50000	50000	**4800**	4800	4800	**2400**	2400	2400	**8000**	8000	**20000**
Distance bow-quay (m)	**7**	7	7	**5**	5	**7**	**4**	4	4	7	7	7
Distance stern-quay (m)	**7**	7	7	**3**	3	**7**	**3**	3	3	7	7	7
Line length bow (m)	**30.5**	30.5	**25**	**25**	25	**30.5**	**25**	25	25	30.5	30.5	30.5
Line length stern (m)	**30.5**	30.5	**25**	**25**	25	**30.5**	**25**	25	25	30.5	30.5	30.5
Line length	**30**	30	**30**	**20**	20	**30**	**30**	30	30	30	30	30

spring (m)												
Nr of lines bow (m)	**2**	2	2	2	2	2	2	2	**3**	2	2	2
Nr of lines stern (m)	**2**	2	2	2	2	2	2	2	**3**	2	2	2
Nr of lines spring (m)	**2**	2	2	2	2	2	2	2	**2**	2	2	2
Axial stress in line (kN)	**177.9**	**300**	177.9	177.9	**50**	50	177.9	**50**	135	**100**	**177.9**	177.9
Elongation, due to axial stress (%)	**8.7**	**9.78**	8.7	8.7	**2.45**	2.45	8.7	**2.45**	8.7	**4.89**	**8.7**	8.7
Natural period (s)	**123.35**	**122.57**	**108**	**29.38**	**29.41**	**36.05**	**22.26**	**22.29**	**22.01**	**46.49**	**46.49**	**73.5**

From the studied variables, the displacement of the vessel influences significant the natural oscillation period of the vessel.
The natural period of the vessel, estimated according to the displacement, without a precise evaluation of the axial stress in the line, does not change notably. If a new evaluation is made, changing the axial stress value (traction in bollard), it results that the differences for the studied cases are about 15% (for small vessels 2400, 4800 tdw) and 1% for the heavy ones (8000, 50000 tdw).
From the above it follows that the natural period is more influenced by changing the line length; for length lines changing of about 20%, the free oscillations periods of the vessel changed 15-20%.

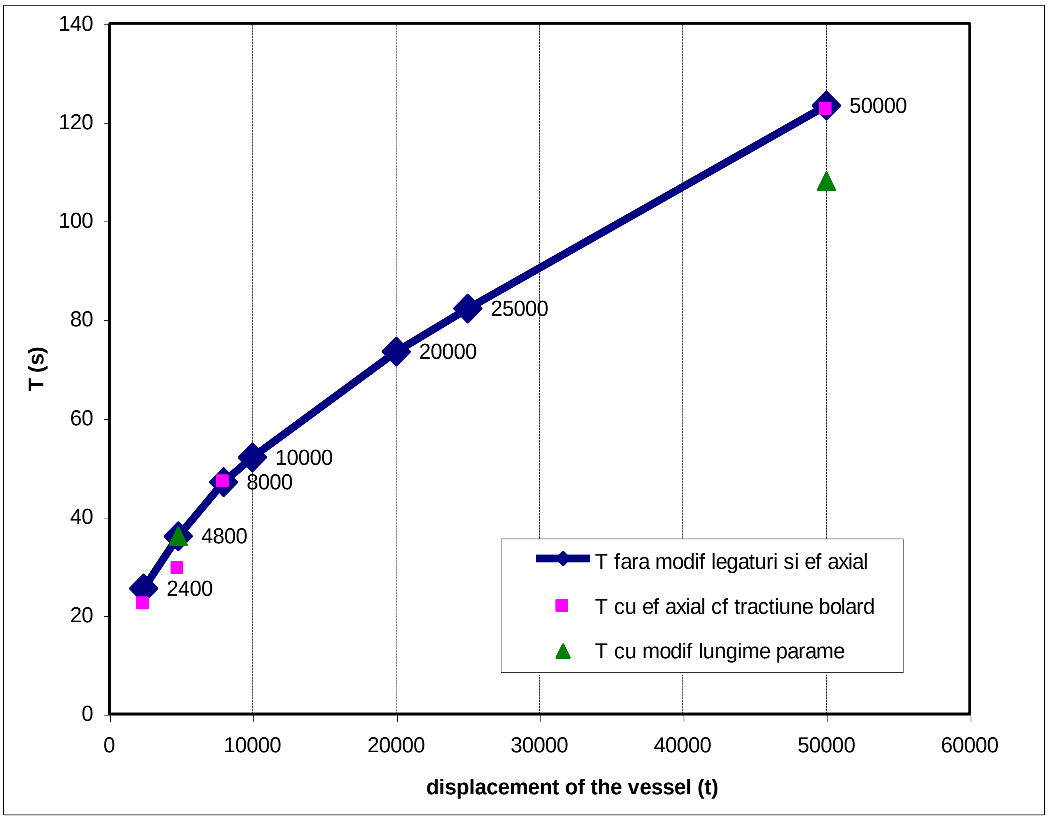

Figure 2. Dependence of the vessel natural oscillation period on its displacement

Two harbour basins were taken into account, serving two berths in Constanta Port:

- B1 - length 600 m, width 300 m and depth 12 m and and allows the mooring of the vessels with a displacement up to 8000 tdw.

- B2 - length 1200 m, width 260 m, depth 17m and allows the mooring of the vessels with a displacement up to 65000 tdw.

If the 8000 tdw vessel moors in the B1 basin, the natural oscillation period of the vessel (46.5 s) is close enough to the oscillation period of the basin, corresponding to the 3rd mode of vibration (44,2 s).
For the B2 basin, designed to be an ore berth, the study of the oscillation period for the design vessel (65000 tdw displacement) leaded to periods of 120.6 s, 126 s, 131 s, 159 s, depending on the length and the number of lines. It can be observed that the oscillation of the vessel with a 65 000 tdw displacement, tied with each 4 lines with a length of 60 m at bow ad stern and 4 springs 65 m long each (for which T=126 s) can resonate with the second harmonic of the basin, characterized by a 124 s period. It must be noticed that the amplification factor is even greater since the entry into resonance occurs for lower vibration modes.
For the same basin, B2, it was identified the risk to resonate with the 2nd mode of vibration also in the case of the 50000 tdw vessel oscillations, if this is tied as presented in case 1.

2. Conclusions

The amplification factor:

- Decreases as the entrance in basin increases, compared to the basin width;
- Decreases for superior natural vibration modes – for the basin completely open on one side. The resonance with the fundamental vibration mode produces an amplification factor of the answer equally with 13, while the resonance with the 3rd vibration mode, the dynamic amplification factor decrease at 7.
- The relative length of the basin that could lead to resonance for any of the 4 studied modes, decreases with the decreasing of the relative width of the entrance in basin

The vessel displacement has a significant influence on the natural oscillation period of the vessel.
If the specific elongation introduced as data in CEDAS is estimated according to the traction in bollard, without changing the line class, there are little differences of the evaluated period:

- 15% for small vessels (2400, 4800 tdw)
- 1% for heavy vessels (8000, 50000 tdw).

The lines length changes the oscillation period: for length changing of about 20% there is a15-20% changing of vessels free oscillations period.
The mooring of the 8000 tdw vessel in the B1 basin, leads to the resonance with the 3rd mode of vibration.
For the B2 basin, designed for an ore berth:

- The oscillations of the 65000 tdw vessel can be produced with the following periods -120.6 s, 126 s, 131 s, 159 s, depending o the lines length and number.
- The oscillations of the 65000 tdw vessel, tied at bow and stern with each 4 lines with a length of 60 m and 4 springs 65 m long each (for which T=126 s) can resonate with the second harmonic of the basin, characterized by a 124 s period – **dangereous situation**
- the amplification factor is even greater since the entry into resonance occurs for lower vibration modes
- there is a risk to resonate with the 2nd mode of vibration also in the case of the 50000 tdw vessel oscillations, if this is tied as presented in case 1.

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Seismic action on mooring front

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Abstract

The study of the dynamic characteristics and response of structures that limit the port waters and have the functionality of mooring construction is very important for the analysis of the moored vessels safety. In the present paper two solutions used to realize the socket were dynamic analyzed: mooring pontoon, consisting of two floating bodies and mooring front made of drilled columns and a canal bank arranged as a slope. The graphic representations allow tracking the ways the high value tangential stress change and the acceleration response of the slope body at various time points of the seismic action, and also how different the distribution is, depending on the absence or presence of drilled columns. Increasing the mooring front design from the port waters allows a more efficient harbor activities.

Rezumat

Studiul caracteristicilor dinamice, precum şi al răspunsului structurilor ce delimitează acvatoriul şi au funcţionalitatea de construcţie de acostare prezintă o mare importanţă pentru comportamentul și analiza siguranţei navelor acostate. În prezenta lucrare s-au analizat din punct de vedere dinamic două soluţii de realizare a alveolei: ponton de acostare, alcătuit din două corpuri plutitoare, respectiv front de acostare alcătuit din coloane forate şi malul canalului amenajat cu taluz. Reprezentările grafice permit urmărirea modului în care se modifică tensiunile tangenţiale cu valori mari şi acceleraţiile de răspuns din corpul taluzului, la diverse momente de timp ale acţiunii seismice, precum şi modul diferit de distribuţie, funcţie absenţa sau prezenţa coloanei forate. Creşterea gradului de robustețe constructivă a fronturilor de acostare din cadrul acvatoriilor permite eficientizarea și continuarea activităţilor portuare în caz de seism.

Keywords: pontoon, drilled columns, dynamic analysis, tangential stress, acceleration

1. Introduction

The study of the dynamic characteristics and response of structures that limit the port waters and have the functionality of mooring construction was performed by 2D finite element numerical modeling using the program COSMOS / M 2.6 with modules for natural frequency analysis and of nonlinear static and dynamic analysis .

Analyzed solutions correspond to the slope wall socket, proposed to be built in the channel margin, so as to achieve efficient port activity.

One of the solutions allows berthing to a pontoon, consisting of two floating bodies. For this situation, the slope structure was analyzed in terms of dynamic.

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The second solution that was dynamically analyzed, was the one in which the front length is proposed to be created out of 10 drilled reinforced concrete columns with a diameter of 0.8 m and inter-distance of 25 m. In this situation, the combination of a column and the canal bank arranged as a slope was dynamically analyzed.
For the modeling of the slope section a triangular finite element with six nodes for plan deformation state was used and the drilled column was modeled with finite element elastic bar type.

2. Dynamic characteristics

For the bank slope to which the floating pontoon is tied to, the following characteristics for the first five vibration modes were obtained (table1), while the presence of the drilled column "stiffened" the slope in some degree, achieving natural periods (table 2):

Table 1

Vibration mode	Pulsation	Period
1	7,23	0,87
2	8,21	0,76
3	9,12	0,69
4	10,20	0,62
5	12,05	0.52

Table 2

Vibration mode	Pulsation	Period
1	7,72	0,81
2	8,92	0,70
3	10,05	0,62
4	11,54	0,54
5	12,19	0,51

3. Dynamic answer to seismic action

Since the consideration of the nonlinearity of material is desired, the dynamic answer must be obtained by the direct integration of motion equations, which in the COSMOS/M program is performed numerically, using nonlinear analysis module with dynamic option.
In this method, the structure's response to the action described by accelerograms is determined for each time step, whilst not necessary to know the natural modes. Disturbing accelerations are assumed to be constant or with a linear variation during the calculation step. Duration of the Δt step must be chosen small enough to achieve satisfactory calculation accuracy and to avoid numerical instability, it is recommended that the step length is at most equal to 1/10 of the smallest value of their natural period which has a significant influence on response. Another criterion in choosing the calculation step refers to the predominant period of dynamic action, recommending 5-10 steps during a semi wave.
It follows that the method requires considerable calculation effort. Dynamic equilibrium conditions must be completed at the beginning and end of each time step. Velocity and displacement calculated at the end of the previous interval are considered as initial conditions for the new time step. The calculation for seismic burdening analysis was conducted with the 10 ms step, and storing the results was performed at 100 ms.

For non-linear dynamic calculation, we used the explicit evaluation of the Rayleigh model damping matrix C = aM+bK. There were proposed the following values : a = 0.6, b = 0.01, which correspond to a critical damping coefficient for the first two modes of vibration of 7.8 %.

For static or dynamic nonlinear analysis it is necessary to define curves of variation of forces over time. Dynamic analysis was performed on the deformed shape obtained under the loading effect of the own weight of the slope and therefore the effect of gravity loading was also introduced into account. We initially made a linear static calculation for the time interval of 0-0.1s, after which we analyzed the nonlinear dynamics, as suggested in figure 1.

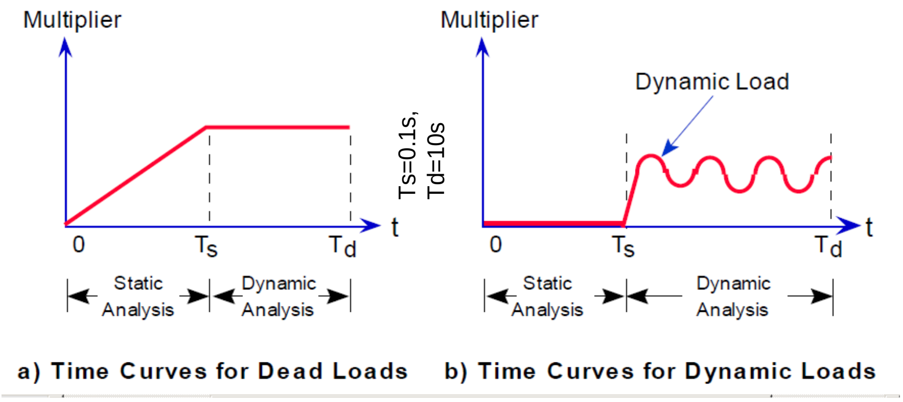

Figure 1 Defining the time variation curves for multipliers of forces in dynamic analysis in which the effect of permanent loads is also considered

Seismic action was modeled as in the figure 2 accelerogram.

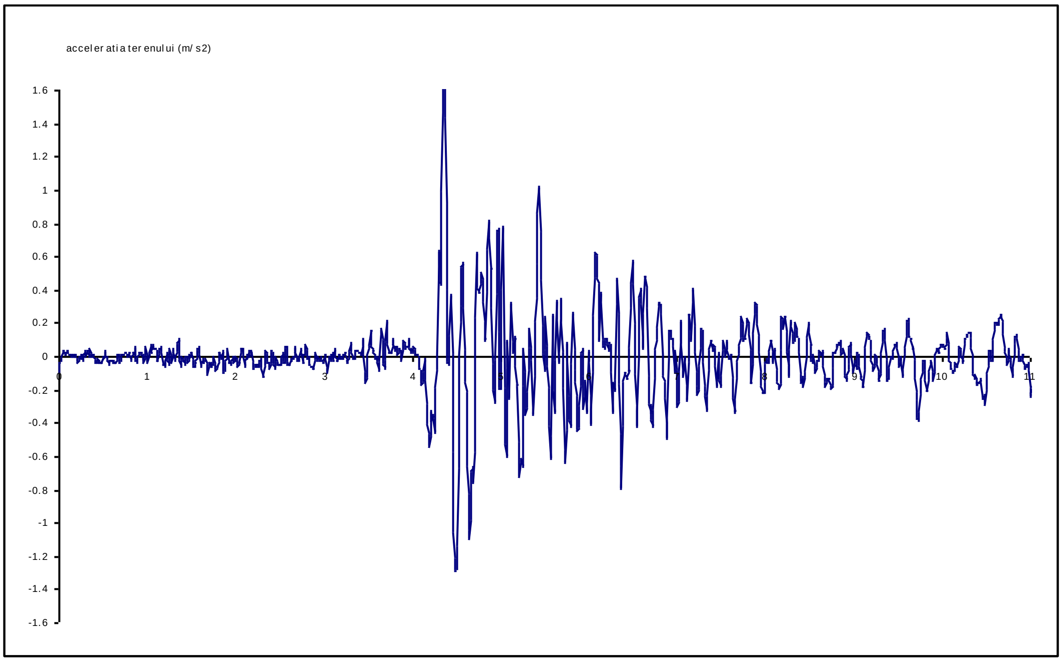

Figure 2 The accelerogram is applied to the base of the structural model

From the dynamic analysis were extracted results illustrating the variation in time or cross-sectional distribution of sizes of defining sizes for the behavior under a dynamic regime, as well as to illustrate the state of tensions in the slope body.

The location of nodes for which time variations were analyzed for both variants - with pontoon and drilled columns, was established.

As sizes expressing the largest dynamic response of the structure we considered movements and horizontal accelerations to be representative, for which data is read in m, respectively m/s^2.

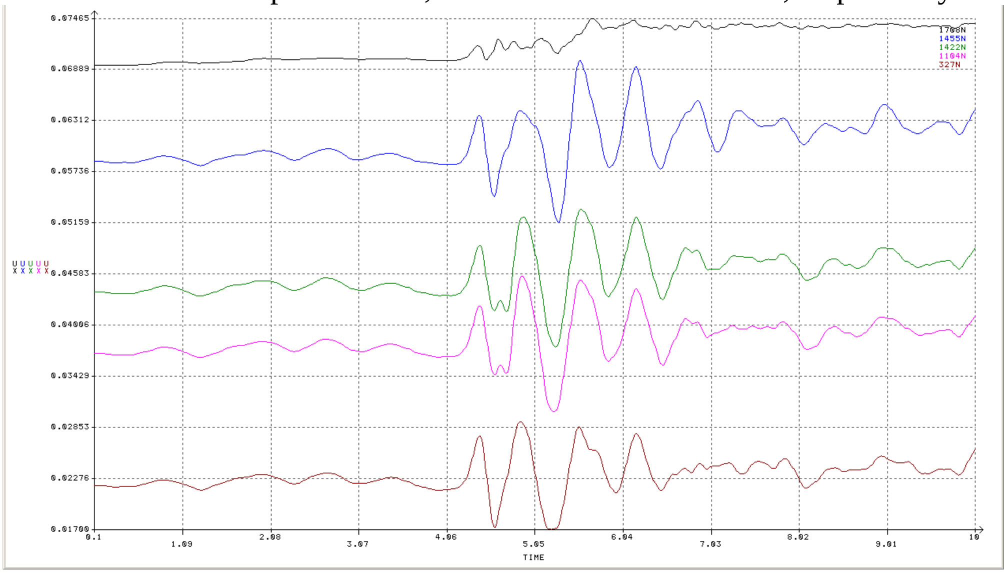

Figure 3. The variation of horizontal translations (m) for nodes 1708, 1455, 1422, 1104, 327 – pontoon solution

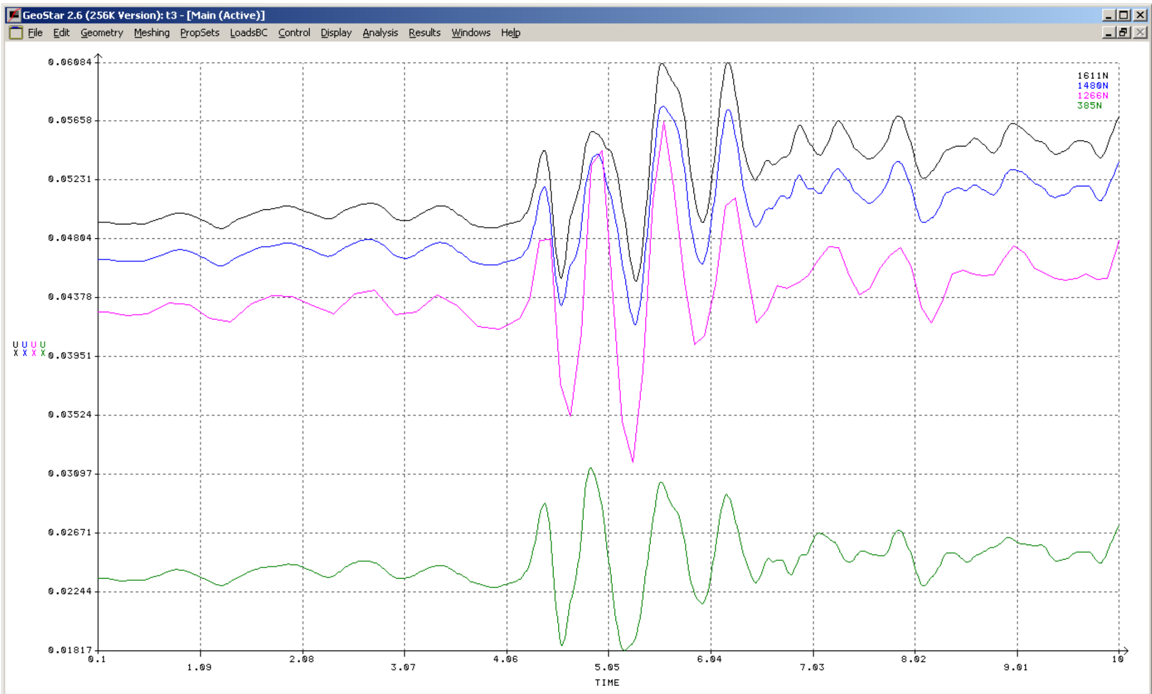

Figure 4. History of response for the horizontal detachment of some nodes with a similar position to Figure 8 drilled column solution

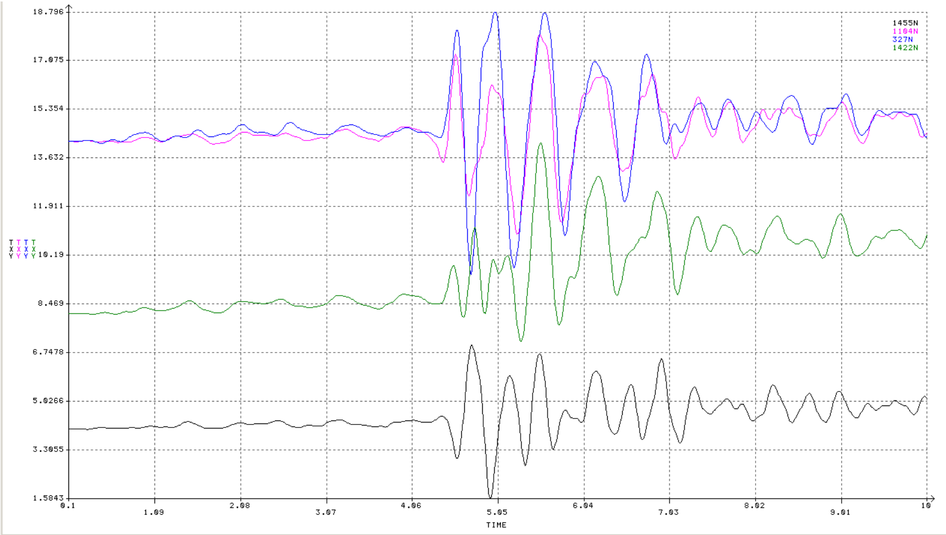

Figure 5. Variation of tangential stress (kN/m^2) for nodes 1455, 1422, 1104, 327 -Pontoon solution

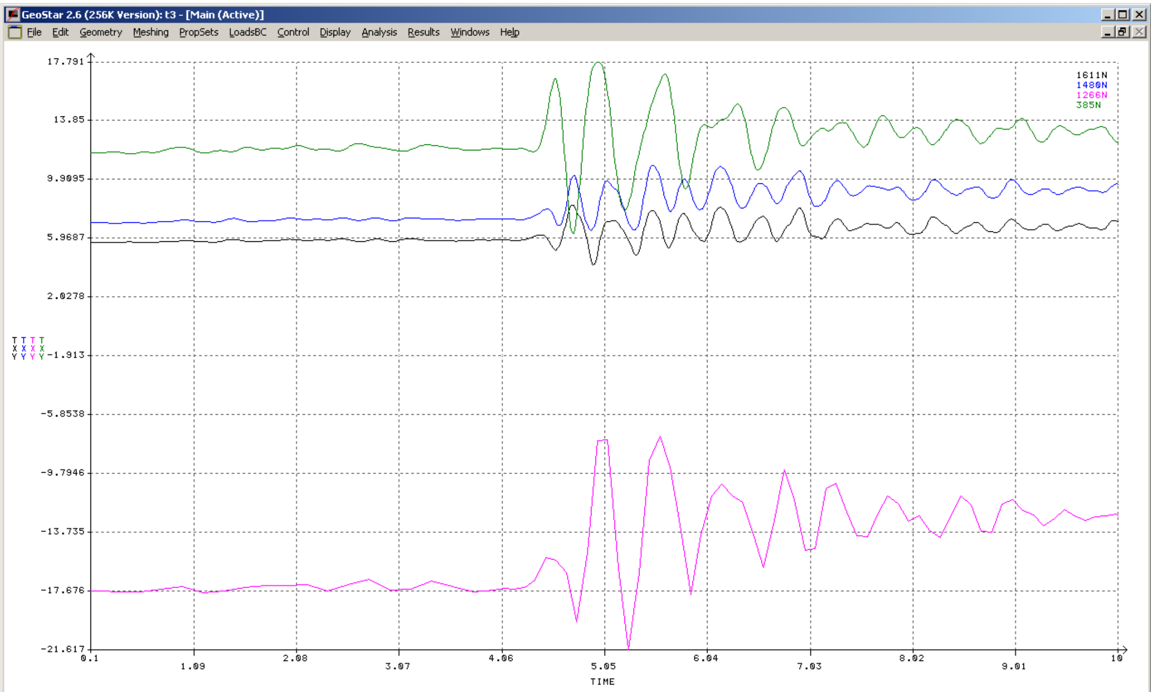

Figure 6. History of response for tangential stress of some nodes with a similar position to Figure 8- drilled column solution

Due to the fact that for the slope construction, stability analysis also involves the analysis of parament sliding stability. An important factor in assessing stability is the quantitative assessment of tangential stress, while both their time evolution and their distribution in the section at various times of time.

Since the slope requirements are complex and are generated in section as normal and tangential stresses, the size reference behind the request is the equivalent stress (σ_{each}). Equivalent stress is corresponding to a simple axial loading (tension or bending) which would produce at the point of calculation in the section the same limit state as the one determined by the reaching of the maximal value of one or other of the corresponding parameters of compound load. The first Rankine's theory of resistance is applied. The criterion underlying the theory is the normal maximum stress. If a state of spacial tension for which $\sigma_1 > \sigma_2 > \sigma_3$, the equivalent stress is the maximum normal stress σ_1. If the tensile and compressive stress limit is denoted by σ_t and σ_{cp}, the resistance condition is expressed through the inequality system $\sigma_{cp} \le \sigma_1(\sigma_2,\sigma_3) \le \sigma_l$ for $\sigma_{cp} < 0, \sigma_l > 0$. Stress values plotted are expressed in kN/m^2.

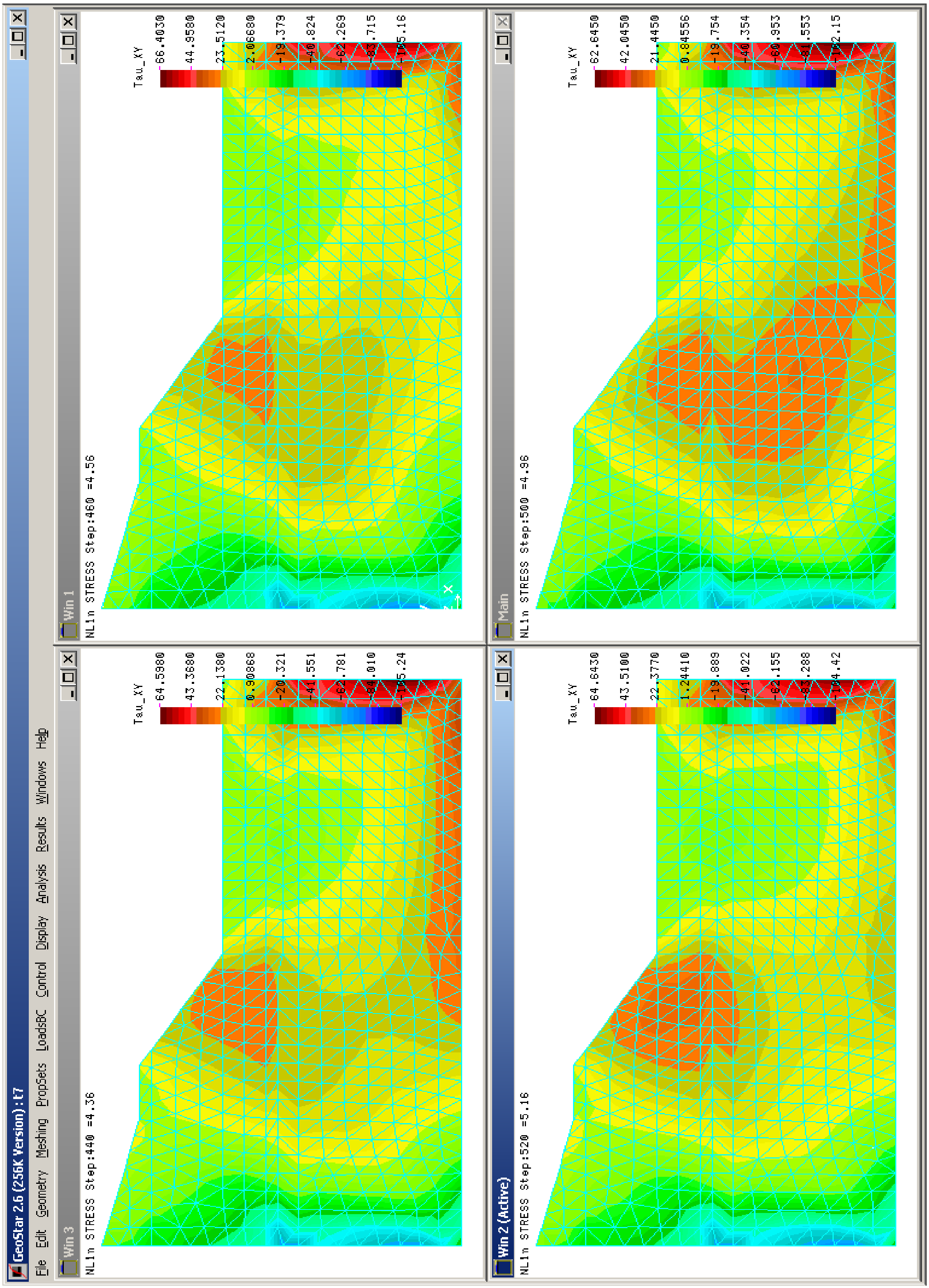

Figure 7. Distribution of tangential tension in the transversal section for the interval 4.20-5 from the debut of the seismic action –pontoon solution

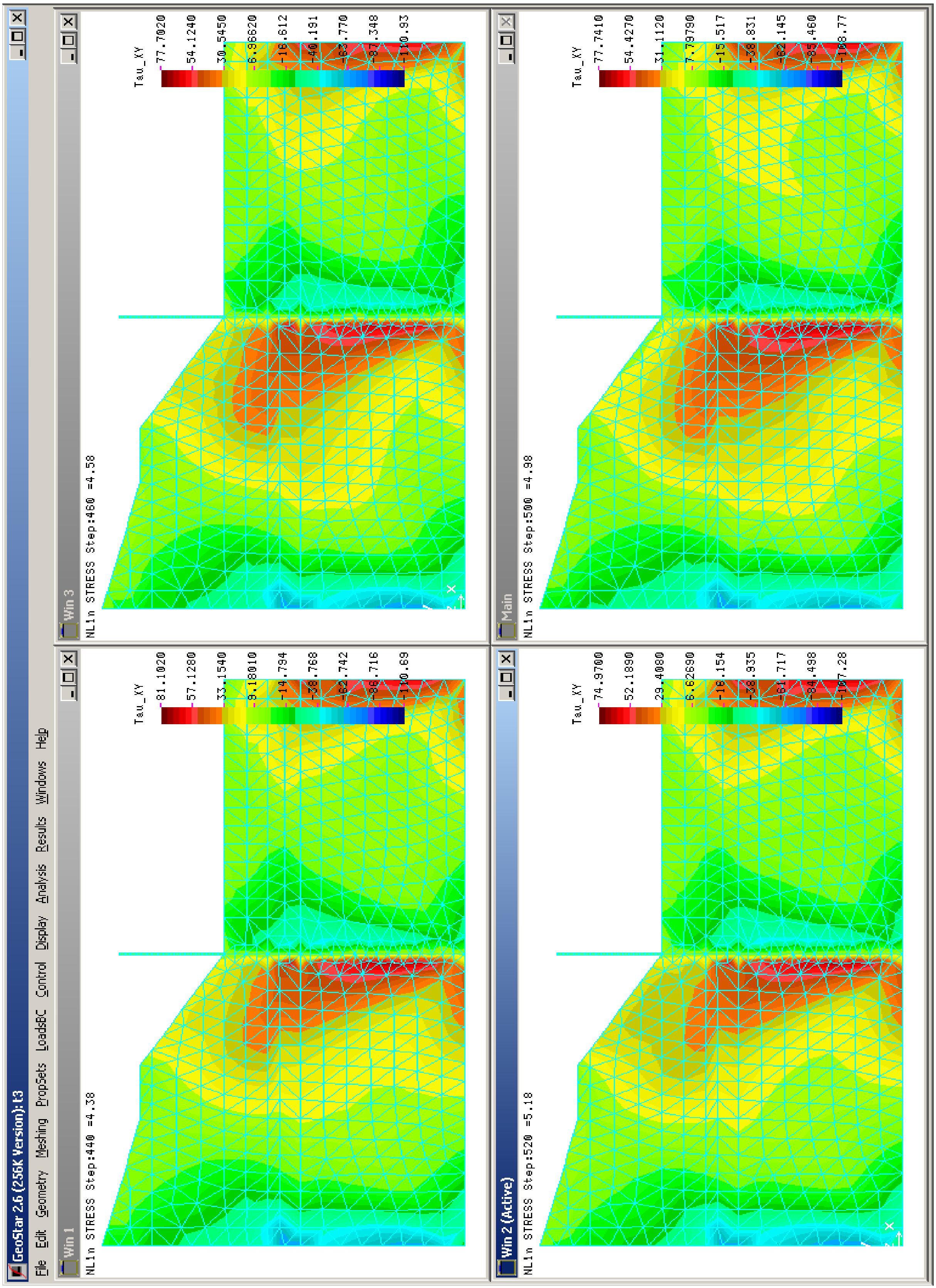

Figure 8. Distribution of tangential stress in a section – drilled column solution

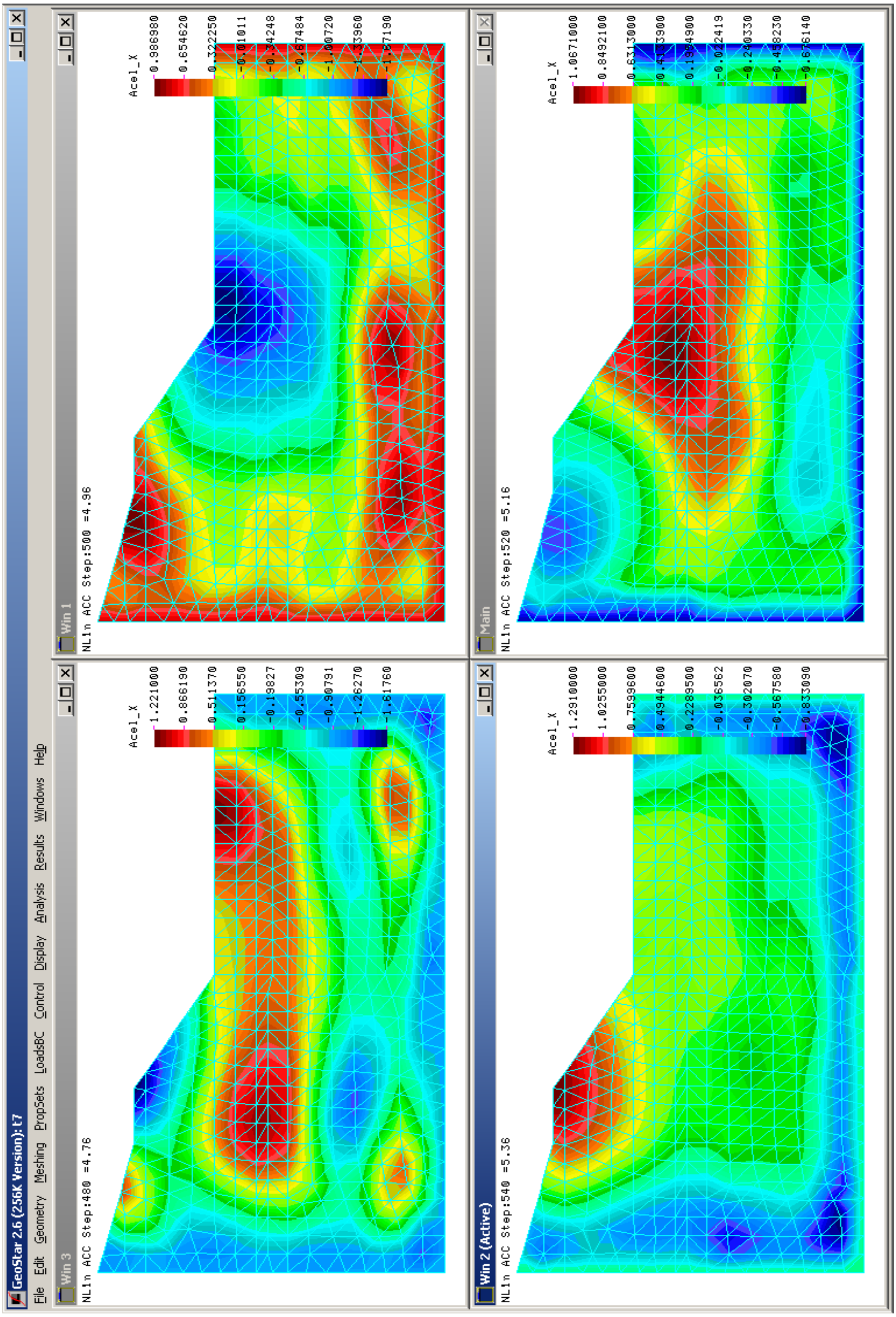

Figure 9. Distribution of accelerations in the slope body, in the transversal section for the interval 4.-5.20s from the debut of the seismic action –pontoon solution

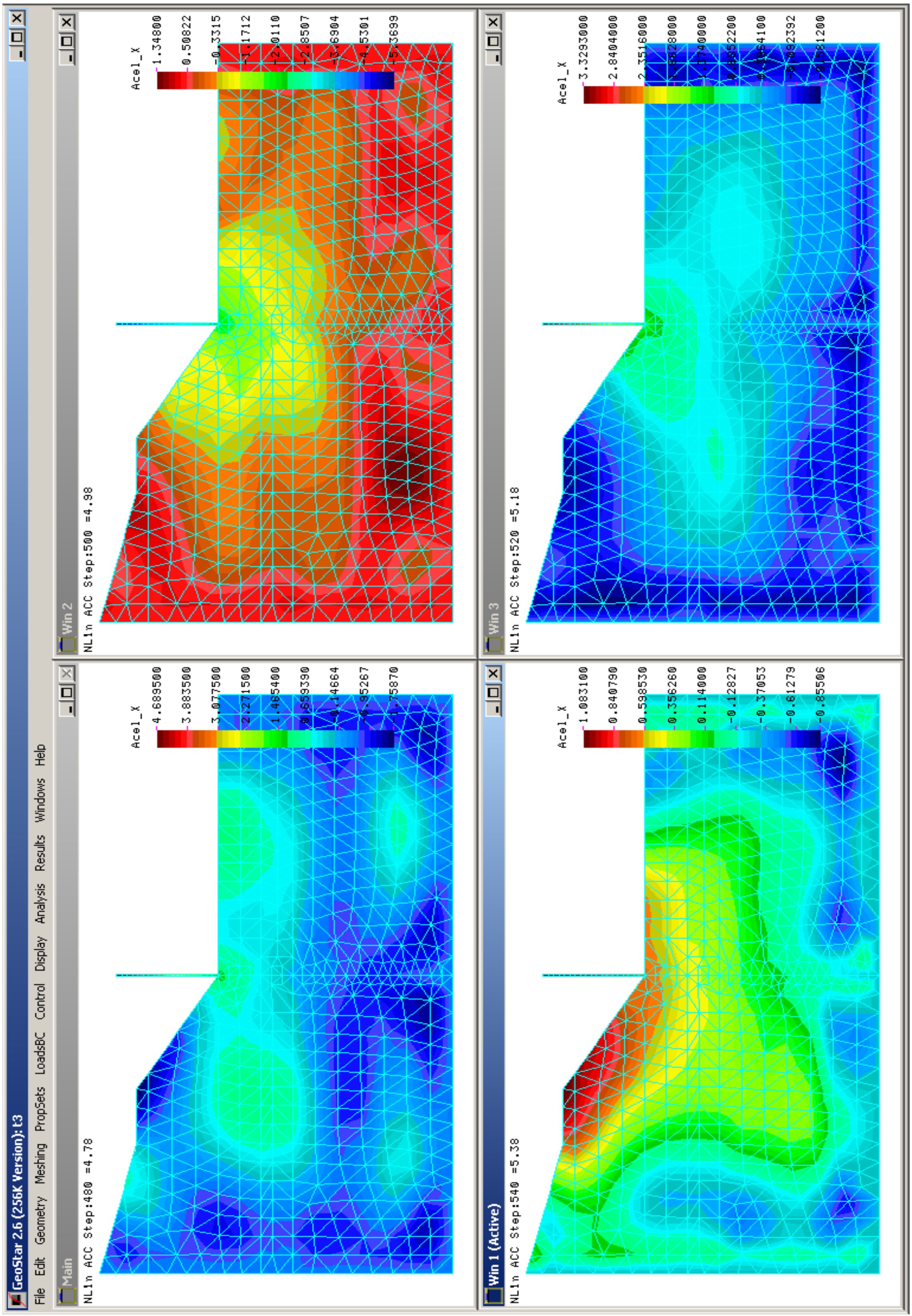

Figure 10. Distribution of horizontal accelerations in a transversal section – drilled column

4. Conclusions

For the comparative analysis of responses from nodes corresponding to the two solutions pairs of

nodes were compared (node from the pontoon solution - node from the drilled columns solution): 1708-1880, 1455-1611, 1422-1480, 1104-1266, 327-385.
It was noted that:
• Displacements and stress due to their own weight are amended by the drilled column presence (for node 1104 - pontoon solution - we obtained a tangent stress from the slope own weight of 14.5 kPa, while in the knot 1266 - the drilled column solution, we obtained negative stress of -17.6 kPa);
• Application of seismic action leads to variations in displacement and stress response for which the maximum amplitude varies, depending on the design solution (for node 1104 - $\frac{18-10.5}{2} = 3.75kPa$ and for node 1266 - $\left|\frac{-21.6-6}{2}\right| = 13.8kPa$).
In the graphical representation shown in Fig.7, Fig.9, Fig.8, and Fig.10, one can notice the tangential stress distribution and the acceleration of the slope body in the cross section for different intervals after the onset of seismic action - for the pontoon, respectively drilled column solution.
The graphic representations allow tracking how shape and the extent of "pockets" of high values of tangential stress, respectively acceleration response of the slope body at various time points of the seismic action, and also how different the distribution is, depending on the absence or presence of drilled columns.
Identifying boundaries between positive and negative tangent stress values indicate hypothetical slip curves, whose form is special in the two cases studied.

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The Influence of Semi-Rigid Connections upon the Performance of Steel Structures Seismically Excited

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Abstract

The paper aims to approach the seismic calculations based on the performance of steel structures with semi-rigid connections, which is a common design issue and which belongs to the state of the art area of structural design. This article deals with this issue from the perspective of a nonlinear geometric analysis and establishing the results in the wider area of performance requisites in connection to seismic activity. The first part aims to develop the theoretical concepts that are needed to include the characteristics of semi-rigid connections in the geometrical nonlinear approach. In the second part, the authors propose a methodology for determining the level of performance for seismically excited steel frames. The two issues mentioned above are integrated by relevant numerical examples. The final part of the paper presents conclusions regarding the effect of such connections on the load bearing capacity and the ductility of the structures from a seismic performance perspective, the advantages and limitations of its usage as a control instrument for the behaviour of the structure under seismic forces.

Rezumat

Lucrarea îşi propune să abordeze calculul seismic pe bază de performanțe al structurilor metalice cu conexiuni semirigide, problemă de proiectare de larg interes şi care se încadrează într-un domeniu de vârf al proiectării structurale. În contextul în care construcțiile devin tot mai ambițioase şi structurile lor tot mai complexe, în acelaşi timp cu creşterea cerințelor vis-à-vis de siguranță, inginerii structurişti se confruntă o situație contradictorie: creşterea capacității portante este asociată, în general, cu reducerea ductilității. Este bine cunoscut faptul că comportarea cadrelor metalice la acțiuni exterioare este în mare măsură influențată de modul de realizare al îmbinărilor stâlp – grindă. Calculele tradiționale au considerat mult timp aceste îmbinărica fiind fie perfect rigide, fie articulații perfecte, analiza fiind dezvoltată pe baza acestei ipoteze. Constatările rezultate în urma unor avarii produse de cutremurele puternice, precum şi concluziile unor studii experimentale recente au arătat că ipoteza nodurilor rigide nu este corectă şi că o analiză corectă a structurilor metalice în cadre trebuie să țină cont de tipul de conexiune şi de caracteristicile mecanice ale acesteia. Articolul tratează acest subiect din perspectiva unei analize geometric neliniare şi încadrând rezultatele în domeniul mai larg al cerințelor de performanță la acțiuni seismice. Vor fi prezentate conceptele teoretice originale pentru includerea în calculul neliniar a caracteristicilor îmbinărilor semi-rigide (relațiile P - D şi M - q_r), exemple numerice relevante, concluzii privind efectul acestor îmbinări asupra capacitătii portante şi a ductilității structurilor din perspectiva performanţei seismice, avantajele şi limitele utilizării lor şi folosirea acestor conexiuni ca instrument de control al comportării structurale sub acțiuni seismice.

Keywords: Performance-based design, steel frames, semi-rigid connections, geometrical nonlinear analysis, finite elements analysis.

1. Analytical model of the bar with semi-rigid connections

In recent years, the civil engineers are faced to a challenging situation: constructions become more and more ambitious and their structures more complex at the same time with the raise in the demand for increased safety measures. This led the structural engineers to face a contradictory situation: the increase in the load bearing capacity is usually associated with the decrease in ductility, the structures becoming sensitive to the seismic action.

The advent of performance-based engineering has placed an emphasis on simulating the nonlinear response of a structural system either to ordinary or seismic action. Accurate and computationally efficient models that represent the nonlinear behaviour in beam-to-column connections are thus required to evaluate the performance of the structures.

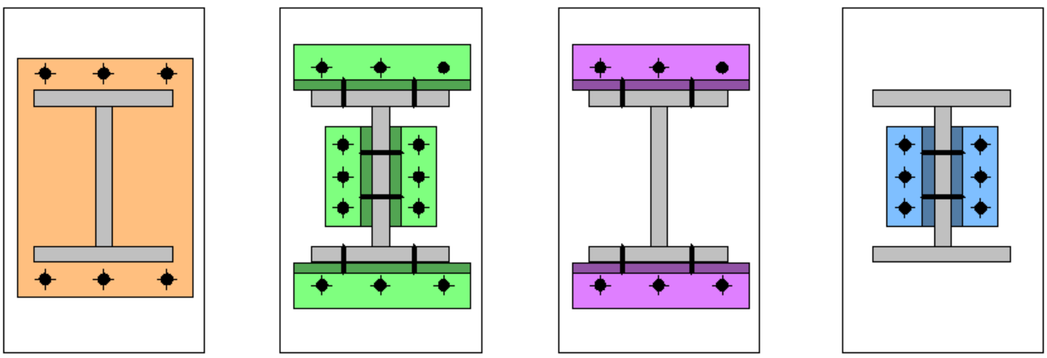
The analysis of the damage produced by strong earthquakes has shown that the hypothesis of rigid nodes is not correct and that an accurate analysis must take into account the real mechanical properties of the beam-to-column connections.

A study on the effects that the Northridge 1994 earthquake had on steel frame-type buildings evidenced the failure of welded beam-to-column connections. Although no collapses of steel structures with welded joints occurred, a large number of cracks in the welding seams of the beam-column connection areas were found, which evidenced the unexpectedly brittle nature of welded connections. The cracks in the welding seams covered a wide spectrum of locations and disclosures. The lesson taught by the Northridge, as well as other strong earthquakes (Kobe, Chi-Chi Taiwan) is not about the economic aspect of post-earthquake rehabilitation, but about the need for really ductile areas at the extremities of the beams in steel frame structures and the conviction that the required ductility level cannot be ensured by welded connections. The measures and provisions regarding the beam-to-column connections of steel frames soon appeared.

Bolted connections are economically and technologically effective, versatile as typology and allow a simple post-earthquake rehabilitation. These connections ensure a high ductility to the beam-column area, which results in high structural ductility.

2. Types of Semi-Rigid Connections

The essential characteristic of semi-rigid connections is the bending moment – relative rotation curve (M - θ_r) in the connection section. These connections, as well as the structure itself, strongly influence the global ductility level of the frames.



(a) (b) (c) (d)

Figure 1. Usual beam-to-column connections: (a) End plate connection; (b) Top and seat + web angle connection; (c) Top and seat angle connection; (d) Web angles connection

Due to this aspect, metal structure designing engineers have focused their attention on the beam-column connection areas, with the technique of bolts instead of welding, on the bearing capacity of these connections under bending moments and the elastic and plastic rotation capacity of the connection area. Thus, we have started to consider the beam-to-column connections of steel frames in a new way: from the stage of an (almost unavoidable) imperfection of execution to the status of standardized connection.

This paper focuses on four usual types of beam-to-column connections, presented in Figure 1. The connection with end plate (Figure 3, a) may be considered as quasi-rigid, the other three connections are semi-rigid with different elastic properties.

2.1 Relationship bending moment – relative rotation

Relationship *M*- q_r is represented through the polynomial model Frye and Morris [1]

$$q_r = C_1(KM)^1 + C_2(KM)^3 + C_3(KM)^5 \tag{1}$$

where K is a standard parameter depending on the geometry of the connection related items and C_i ($i = 1,2,3$) are dimensionless parameters that depend on the type of the connection. This model covers a great variety of usual beam-to-column connections and numerical experiments showed a good concordance to experimental results. The geometrical parameters for the connections presented in Fig. 1 are synthesized in Table 1. The *M*- q_r curves for the studied types of connections are represented in Fig. 2.

Table 1. Geometrical parameters of the connection

beam	column	K
IPE0550+ t_w=t_i=12,7*mm* b=212*mm* h_i=467,6*mm* t_f=t_t=20,2*mm*	**HE500B** t_i=14,5*mm* b=300*mm* h_i=500*mm* t_t=28*mm*	$K = d_g^{-2.4}\, t_p^{-0.4}\, d_b^{-1.5}$ K=0,000008725
angles	bolts	
L100x100x10 d_a=300*mm* t_a=10*mm* g=110*mm*	*M*24	$C_1 = 1.83 \times 10^{-3}$, $C_2 = -1.04 \times 10^{-4}$, $C_3 = 6.38 \times 10^{-6}$
$q_r = 15{,}967 \cdot 10^{-9} M - 690{,}76 \cdot 10^{-22} M^3 + 28145{,}78 \cdot 10^{-35} M^5 \quad [rad]$		

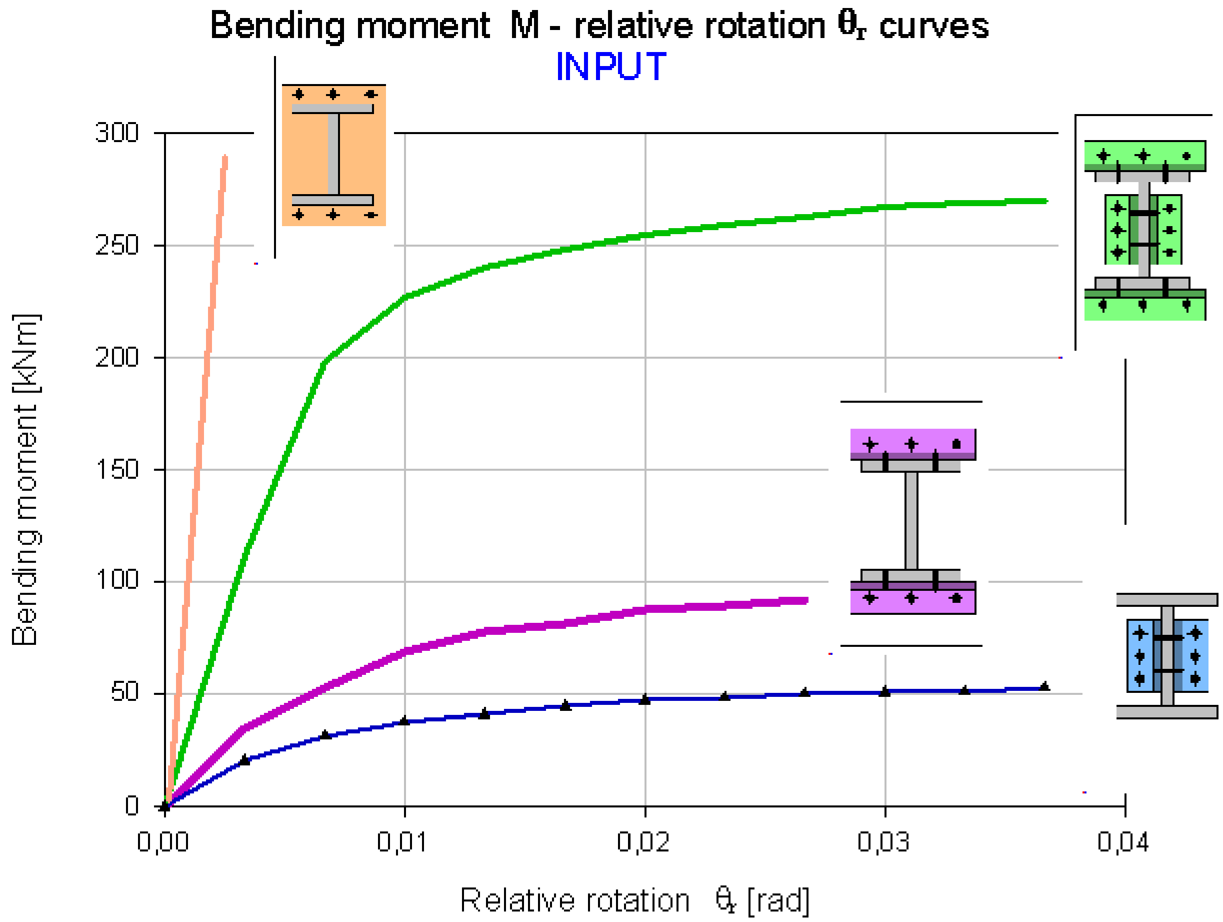


Figure 2. Curves M- θ_r for the studied types of connections

2.2 Geometrical nonlinear finite element model for the bar with elastic connections

The static analysis is formulated through a finite element approach. The FEM model for the bar with elastic connections is represented in Fig. 3.

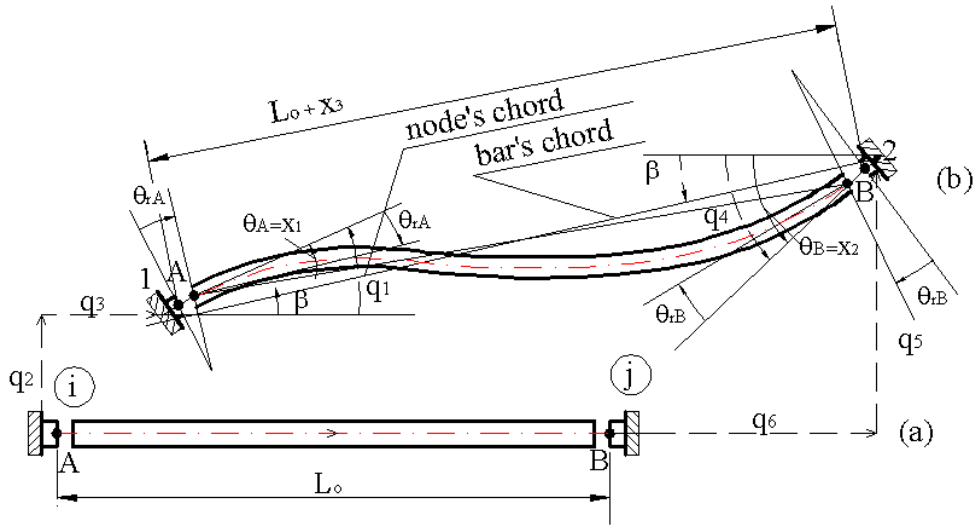


Figure 3. Finite element for the bar with elastic connections at the ends

Fundamental vectors associated to the finite element are: nodal displacements $\boldsymbol{q}$, nodal forces $\boldsymbol{Q}$, internal actions $\boldsymbol{X}$ and strain vector $\boldsymbol{x}$. Equilibrium equations (2) are expressed on the deformed structure and the boundary conditions are (3):

$$\boldsymbol{Q} = \boldsymbol{A}^T \boldsymbol{X} \tag{2}$$

$$\boldsymbol{x} = \boldsymbol{A}\boldsymbol{q} \tag{3}$$

Matrix $\boldsymbol{A}$ enclose the nonlinear character through the influence of the nodal displacements $\boldsymbol{q}$ upon the strains at the ends of the element $\boldsymbol{x}$. In addition, it contains also the influence of the finite dimensions of the joints and deviations from the axis:

$$\boldsymbol{A}^T = \begin{bmatrix} 1+\dfrac{a_x}{\overline{L}} & \dfrac{a_x}{\overline{L}} & 0 \\ \dfrac{1}{\overline{L}} & \dfrac{1}{\overline{L}} & -\dfrac{q_4-q_5}{\overline{L}} \\ \dfrac{q_2-q_5+a_xq_1+b_xq_4}{\overline{L}^2} & \dfrac{q_2-q_5+a_xq_1+b_xq_4}{\overline{L}^2} & -\dfrac{L}{\overline{L}} \\ \dfrac{b_x}{\overline{L}} & 1+\dfrac{b_x}{\overline{L}} & 0 \\ -\dfrac{1}{\overline{L}} & -\dfrac{1}{\overline{L}} & -\dfrac{q_4-q_5}{\overline{L}} \\ -\dfrac{q_2-q_5+a_xq_1+b_xq_4}{\overline{L}^2} & -\dfrac{q_2-q_5+a_xq_1+b_xq_4}{\overline{L}^2} & \dfrac{L}{\overline{L}} \end{bmatrix} \tag{4}$$

where a_x, b_x, a_y and b_y are the eccentricities of nodes 1 and 2 with respect to the connecting sections A, B and

$$\overline{L} = L_0 + q_6 - q_3 \tag{5}$$

Constitutive law of the material for a bar with elastic connections are:

$$\mathrm{d}\boldsymbol{X} = \boldsymbol{k}\mathrm{d}\boldsymbol{x} \tag{6}$$

Because deformations x_1 and x_2 include the relative rotations θ_{rA} and θ_{rB} due to elastic connections at the ends of the bars it result:

$$\begin{aligned} \mathrm{d}X_1 &= \frac{4EI}{L}(\mathrm{d}x_1 - \mathrm{d}\theta_{rA}) + \frac{2EI}{L}(\mathrm{d}x_2 - \mathrm{d}\theta_{rB}) \\ \mathrm{d}X_2 &= \frac{2EI}{L}(\mathrm{d}x_1 - \mathrm{d}\theta_{rA}) + \frac{2EI}{L}(\mathrm{d}x_2 - \mathrm{d}\theta_{rB}) \\ \mathrm{d}X_3 &= \frac{EA}{L}\mathrm{d}x_3 \end{aligned} \tag{7}$$

If R_{iA} and R_{iB} is the initial stiffness at the end of the bar, the static-kinematic duality yields:

$$\mathrm{d}\theta_{rA} = \frac{\mathrm{d}X_1}{R_{iA}} \qquad \mathrm{d}\theta_{rB} = \frac{\mathrm{d}X_2}{R_{iB}} \tag{8}$$

and stiffness matrix of the element is

$$\boldsymbol{k} = \begin{bmatrix} \dfrac{4EIR_{iA}R_{iB}L + 12(EI)^2 R_{iA}}{(R_{iA}L + 4EI)(R_{iB}L + 4EI) - 4(EI)^2} & \dfrac{2EIR_{iA}R_{iB}L}{(R_{iA}L + 4EI)(R_{iB}L + 4EI) - 4(EI)^2} & 0 \\ \dfrac{2EIR_{iA}R_{iB}L}{(R_{iA}L + 4EI)(R_{iB}L + 4EI) - 4(EI)^2} & \dfrac{4EIR_{iA}R_{iB}L + 12(EI)^2 R_{iB}}{(R_{iA}L + 4EI)(R_{iB}L + 4EI) - 4(EI)^2} & 0 \\ 0 & 0 & \dfrac{EA}{L} \end{bmatrix} \tag{9}$$

Accepting a finite formulation, the geometric nonlinear stiffness matrix of the bar with elastic connections is:

$$\boldsymbol{K} = \boldsymbol{A}^T \boldsymbol{k} \boldsymbol{A} \tag{10}$$

The theory was validated by numerical studies on structures recognized for the highly nonlinear nature of their behaviour as William's Toggle and Diamond Shaped Frame [2].

3. Performance criteria for steel structures

Large economic losses and loss of function for buildings and facilities of vital importance as well as causalities following a major seismic event (Northridge 1994, Kobe 1995, Taiwan 1999) determined civil engineers to approach seismic design from a new perspective.

In the 1990s, Federal Emergency Management Agency (FEMA) develops the first document in which essential earthquake-related concepts are articulated to a performance-based procedure [3]. The key concept is a performance objective, consisting in a design event which the building is design to resist and an accepted level of damage (performance level) in the case that the design event is experienced.

The European design code [4] contains two fundamental requirements for buildings: (1) no collapse requirement and (2) damage limitation requirement. The first requirement is associated, according to the National Annex [5] to a seismic action with the probability of exceedance of 39% in 50 years (reference return period 100 years), while for the second requirement, the seismic event has the probability of exceedance of 28% in 10 years (reference return period 30 years). Eurocode 8 (2006) does not specify accepted degradations for structural or non-structural elements and, practically does not introduce performance levels for buildings. From this perspective, U.S. regulations are more explicit [6], [7].

The seismic performance of a structure may be stated, according to the authors, on two main criteria: (1) collapse prevention and (2) occupancy during/after the seismic event. Based on these criteria, in Figure 4 are proposed four levels of performance which establish the extension of the damage allowed after the earthquake. The parameter chosen to measure the performance of the building is the relative drift D_r, calculated as the ratio between the displacement at the top and the height of the building. Performance levels proposed in Fig. 4 and defined in Table 2 are a synthesis of both American and European regulations mentioned above.

Table 2. Performance levels and severity of damage for steel frames

Performance level	Maximum relative drift [%]	Damage description	Downtime
Fully operational (F.O.)	0,2	Negligible structural damage	-
Operational (O.)	0,5	Light structural damage: - minor local yielding or buckling - no fractures Structure safe for occupancy	24 hours
Life protection (L.P.)	1,5	Moderate structural damage: - plastic hinges form - local buckling of beams - joint distortions - isolated failure in connections Structure remains stable	Possible partial loss (Repair possible but may be economically impractical)
Imminent collapse (I.C.)	2,5	Severe structural damage - extensive distortions in beams - extensive failure of connections Structural collapse prevented, possible restricted access, non-structural elements may fail	Possible total loss

Fig. 4 illustrates the nonlinear procedure for performance assessment of the structure. A nonlinear model of the structure is subjected to monotonically increasing force to create the force – displacement curve, namely the capacity curve of the structure.

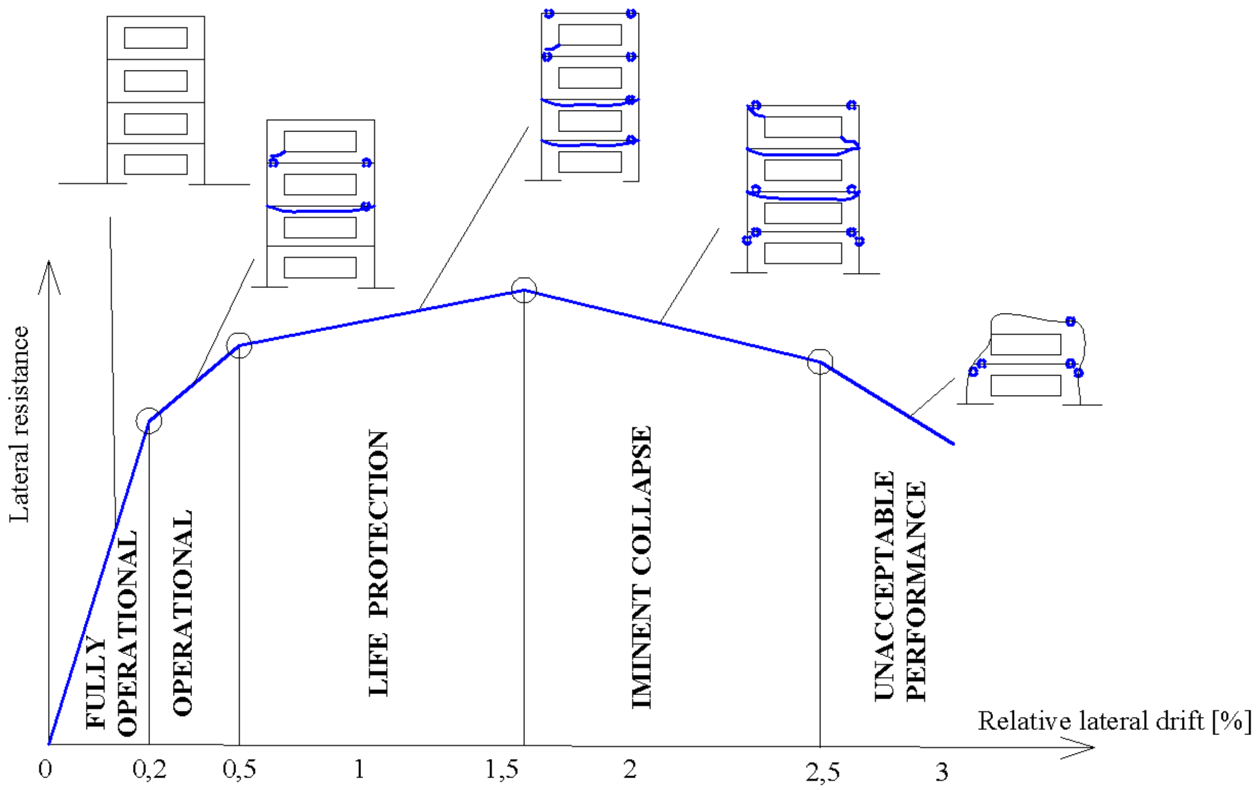


Figure 4. Proposed performance levels for steel frames

The level of performance required by a structure is assigned according to: (1) the importance of the building and (2) the probability of occurrence of the seismic event (Fig. 5).

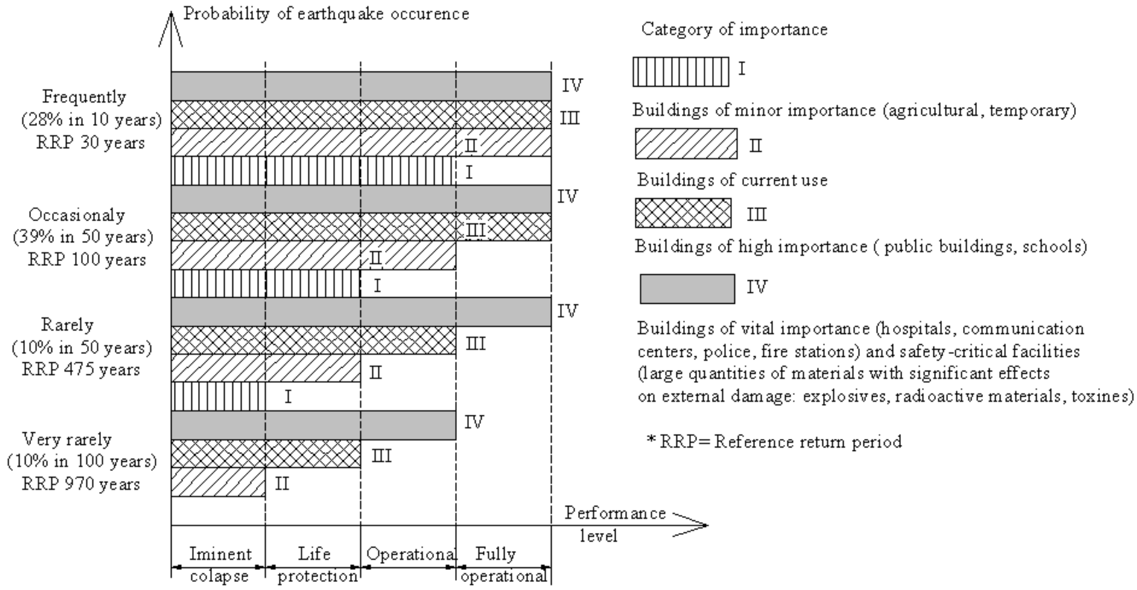


Figure 5. Performance levels and earthquake probability

4. Numerical results

Since Northridge and Kobe earthquakes, a pushover analysis becomes an accepted and simple method for the seismic evaluation of structures. The purpose of the pushover analysis is to evaluate the expected performance of a structure by estimating its strength and deformation demands in design earthquakes by means of a nonlinear analysis, and comparing these demands to available capacities at the performance levels of interest. The nonlinear pushover analysis, or collapse mode analysis, is relatively simple technique, but very efficient to capture the essential features that significantly affect the seismic performance goal.

The theory was applied to multi-storey frames with various equipping in beam-to-column connections [8]. Numerical computations was performed with the program ABAQUS that allows direct, easy and versatile modeling of a wide range of connections.

The article presents numerical results for three frames: (1) 10 levels, 1 span; (2) asymmetrical 10+5 levels and (3) 10 levels, 2 spans. For each case, the beam-to-column connections are accomplished in the four options shown in Fig. 1. Loading consists on a constant gravitational force applied to the end of the beams and a lateral pushover-type force controlled by the loading parameter P. The results presented in Figures 6, 7 and 8 refer to the curves load P– relative lateral displacement at the top D_r.

The numerical results presented in Fig. 6 – 8 lead to a general conclusion, namely that the relationship $P – D_r$ is strongly influenced both by the type of connection and the associated M- q_r analytical model. For the four types of semi-rigid connections considered in the analysis is established, a similar reaction: a linear quasi-rigid relationship $P – D_r$ for low levels of loading is followed by a strong relaxation in the connections for high loading levels.

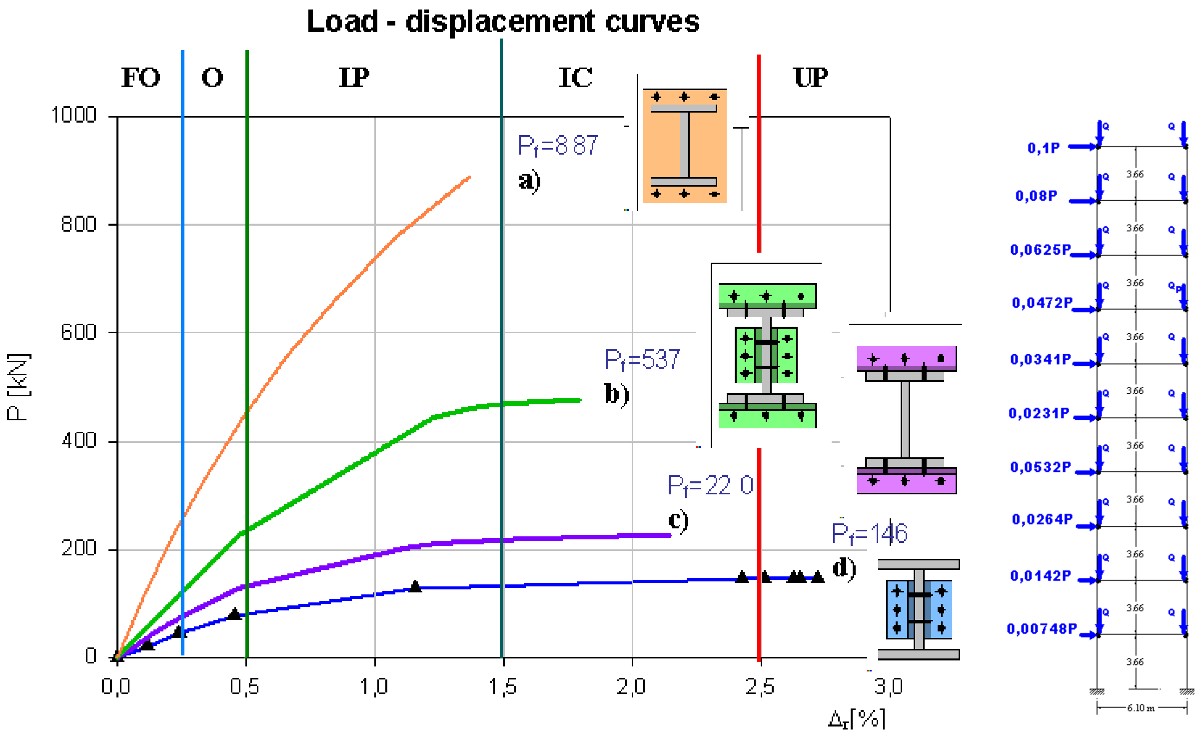


Figure 6. Structure (1) – 10 levels, 1 span.

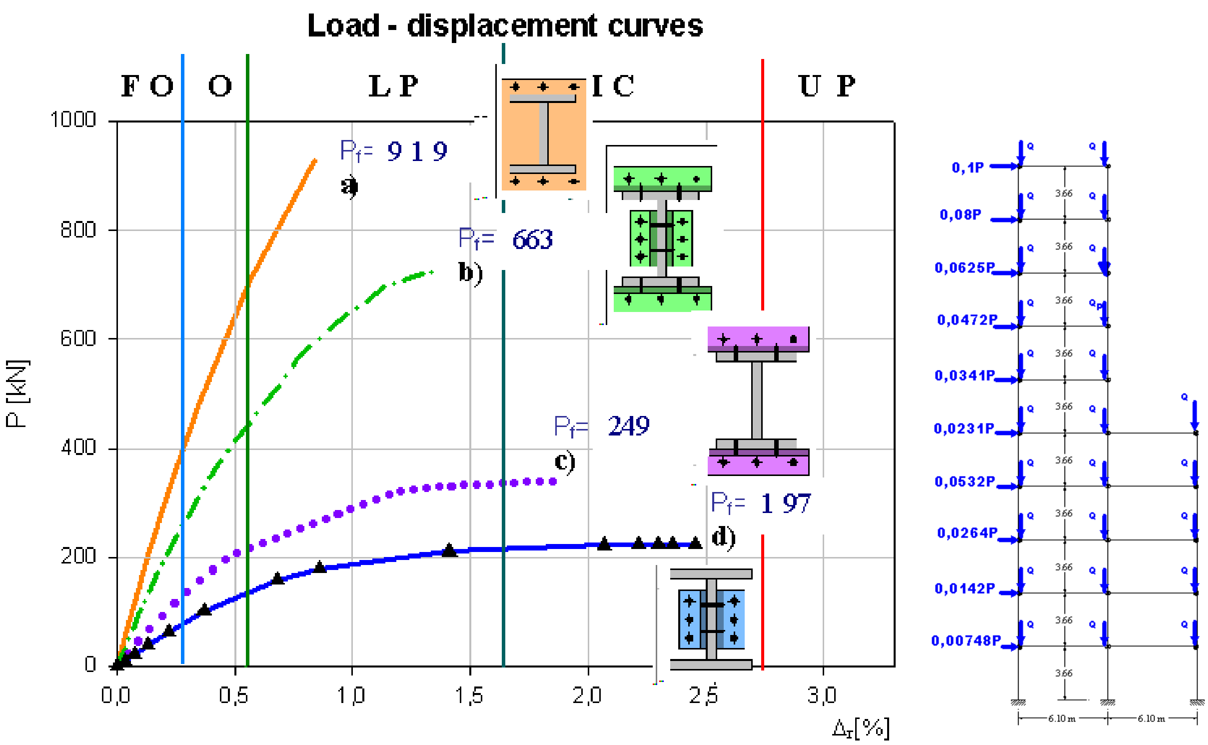


Figure 7. Structure (2) – 10 + 5 levels.

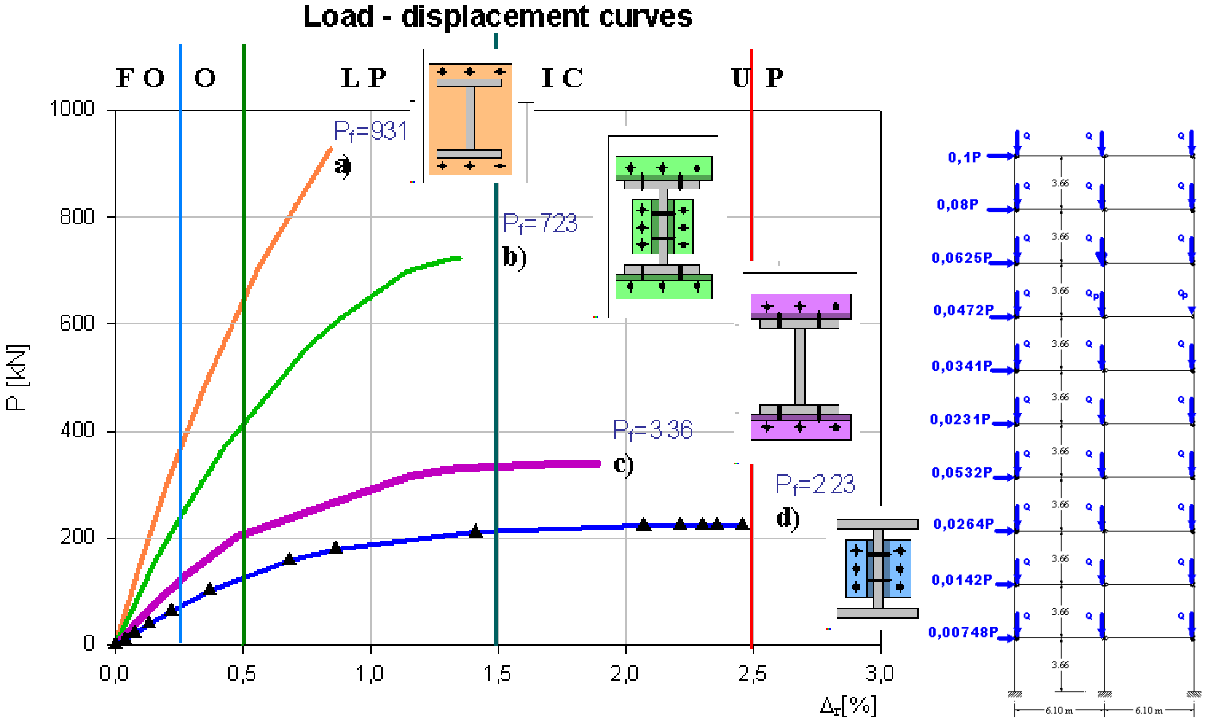


Figure 8. Structure (3) – 10 levels, 2 spans.

Non-linearity of P – D$_r$ curves for the structures studied suggests a pseudo-plastic behaviour, assuming the strain energy dissipation in the joint sections. Yielding of connecting elements instead of beams, columns or welding seams, is technically an affordable way and technologically an efficient solution of achieving performing steel structures in seismic regions.

Table 3. Failure force and relative drift at the top

Structure	Type of connection	Failure force		Relative drift	
		P_f [kN]	%	Δ_r	%
10 levels 1 span	a b c d	887 537 220 146	100 60,54 24,80 16,46	1,360 1,800 2,130 2,720	100 132,35 156,61 200,00
10+5 levels 2 spans	a b c d	919 663 249 197	100 72,14 27,09 21,43	1,090 1,456 2,036 2,689	100 133,57 186,70 246,70
10 levels 2 span	a b c d	931 723 336 223	100 77,65 36,09 23,95	0,884 1,360 1,863 2,480	100 153,83 210,74 280,54

Analysis of displacements shows that the need of ductility can be obtained through the semi-rigid behaviour of the beam-to-column connections. The structures with semi-rigid connections present a lateral resistance between 78% … 16% compared to the quasi-rigid connection (Table 3).

The bearing capacity of the frames reduces with the flexibility of the beam-to-column connection and the height of the structure. Ductility levels, associated to the elasto-plastic biography, expressed by the relative displacements at the top, increase with 32% … 180% for the semi-rigid beam-to-column connection comparative to the rigid one.

The results obtained illustrate once again that the drift of the structure is closely related to the rotation (elastic and inelastic) demands on individual beam-column connections, and therefore, is closely related to the performance level of the structure.

5. Conclusions

The assessment of the performance level, for the structures analyzed in the previous paragraph, is synthesized in Table 4. Evaluation of the seismic load was made by Romanian standards [4], [5], the earthquake took into account has a return period of 100 years and can be considered, by frequency of occurrence, as occasionally.

Pushover analysis reveals that collapse of steel frames with semi-rigid connections occurs generally after depletion of life protection resources, while rigid structures are exhausted before this range. Also must be pointed out that excessive flexibility of the beam-to-column joints leads to a dramatic reduction in the overall strength resistance of the steel frames.

The results obtained highlight once again the two contradictory effects of the semi-rigid connections on seismically excited steel frames: (1) the positive effect consisting in significantly improved ductility and thus gaining of a higher level of performance and (2) the negative effect manifested by significant reduction in resistance. The two effects cannot be separated: the earthquake-induced loads just need ductility.

Table 4. Performance Levels for the Analyzed Structures

Type of structure	Structure (1) 10 levels 1 span		Structure (2) 5=10 levels 2 spans		Structure (3) 10 levels 2 spans	
Seismic zone:	Cluj-Napoca, Romania		a_g=0,08g; T_c=0,7 sec			
	F_b=96,80kN		F_b=63,07kN		F_b=178,41kN	
Performance level	a	F.O.	a	F.O.	a	F.O.
	b	F.O.	b	F.O.	b	F.O.
	c	O.	c	O.	c	O.
	d	L.P.	d	L.P.	d	L.P.
	a	Buildings of vital, high importance, current use and minor importance	a	Buildings of vital, high importance, current use and minor importance	a	Buildings of vital, high importance, current use and minor importance
Category of building	b		b		b	
	c	Buildings of current use and minor importance	c	Buildings of current use and minor importance	c	Buildings of current use and minor importance
	d	Buildings of minor importance	d	Buildings of minor importance	d	Buildings of minor importance

The lessons taught by the Northridge and Kobe earthquakes, and other recent major seismic events, tilt the positive – negative balance towards the positive effect of the semi-rigid connections. Besides that, semi-rigid beam-to-column connections make post-earthquake rehabilitation more easily achievable in financial terms. Semi-rigidity should be considered as an effective solution to meet performance criteria required for buildings in seismic areas.

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A Finite Element Approach for Plane Steel Structures with Semi-rigid Connections

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Abstract

It is well known that the behaviour of steel structures is highly influenced by the way in which the beam-to-column connection is accomplished. This article deals with this issue from the perspective of a nonlinear geometric analysis of frame structures in the case of semi-rigid beam-to-column connections. The analysis itself is consistent, using the finite element method applied to structures consisting of straight bars. The static and kinematical parameters specific for geometrically nonlinear analysis are synthesized in vector form, and fundamental relationships are inferred by the separate and independent introduction of static, kinematical and constitutive conditions to be met. The issues mentioned above are integrated by relevant numerical examples. The final part of the paper presents conclusions regarding the effect of the semi-rigid connections on the load bearing capacity and the ductility of the structures.

Rezumat

Este bine cunoscut faptul că răspunsul structurilor metalice la acţiuni exterioare este puternic influenţat de modul de realizare al îmbinării sâlp – grindă. Calculele tradiţionale au considerat mult timp aceste îmbinări ca fiind fie perfect rigide, fie articulaţii perfecte, analiza fiind dezvoltată pe baza acestei ipoteze. Articolul îşi propune abordarea acestui subiect din perspectiva unei analize geometrice neliniare aplicată structurilor plane având îmbinări semirigide stâlp – grindă. Analiza în sine utilizează metoda elementelor finite aplicată structurilor plane alcătuite din bare drepte. Sunt prezentate conceptele teoretice originale necesare includerii în analiza cu elemente finite a caracteristicilor îmbinărilor semirigide. Parametrii statici şi cinematici specifici calculului geometric neliniar sunt sintetizaţi în formă vectorială iar condiţiile statice, cinematice şi constitutive sunt introduce independent. Aspectele menţionate mai sus sunt integrate prin exemple numerice relevante. Partea finală a articolului conţine concluzii referitoare la efectul semirigidităţii îmbinărilor stâlp – grindă asupra capacităţii portante şi a ductilităţii structurilor metalice.

Keywords: Plane steel structures, connections, semi-rigidity, geometrical nonlinear analysis, finite element analysis.

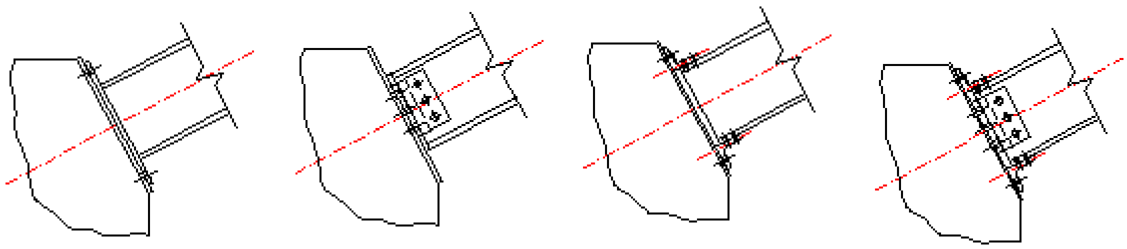
1. Analytical model of the semi-rigid beam-to-column connection.

The analysis of the damage produced by strong earthquakes, as well as the conclusions of several recent experimental studies have shown that the hypothesis of the rigid nodes is incorrect and that an accurate analysis of steel structures must take into account the type of connection and its mechanical characteristics.

1.1 Types of semi-rigid connections

The essential characteristic of semi-rigid connections is the bending moment – relative rotation curve (M - θ_r) in the connection section. The seismic performance of steel frames depends to a large extent on the elastic properties of the beam-to-column connections. These connections, as well as the structure itself, strongly influence the global ductility level of the frames. Due to this aspect, metal structure designing engineers have focused their attention on the beam-column connection areas, with the technique of bolts instead of welding, on the bearing capacity of these connections under bending moments and the elastic and plastic rotation capacity of the connection area. Thus, we have started to consider the beam-to-column connections of steel frames in a new way: from the stage of an (almost unavoidable) imperfection of execution to the status of standardized connection.

This paper focuses on four usual types of beam-to-column connections, presented in Fig. 1. The connection with end plate (Fig. 1, a) may be considered as quasi-rigid, the other three connections are semi-rigid with different elastic properties.



a. b. c. d.

Figure 1. Usual beam-to-column connections:

a. end plate; b. two web angles; c. two flange angles; d. four angles (two web and two flange angles)

1.2 Relationship bending moment – relative rotation

Relationship M- θ_r is represented through the polynomial model Frye and Morris [1]

$$\theta_r = C_1(KM)^1 + C_2(KM)^3 + C_3(KM)^5 \tag{1}$$

where:

$-K$ is a standard parameter depending on the geometry of the connection related items;

$-C_i$ $(i = 1,2,3)$ are dimensionless parameters that depend on the type of the connection.

This model covers a great variety of usual beam-to-column connections and numerical experiments showed a good concordance to experimental results. The geometrical parameters for the connections presented in Figure 1 are synthesized in Table 1. The M- θ_r curves for the studied types of connections are represented in Fig. 2.

Table 1. Geometrical parameters of the connection

beam	column	angles	bolts	K
IPE0550+ t_w=t_i=12,7*mm* b=212*mm* h_i=467,6*mm* t_f=t_t=20,2*mm*	**HE500B** t_i=14,5*mm* b=300*mm* h_i=500*mm* t_t=28*mm*	End plate d_g=660*mm* t_p=10*mm* d_b=24*mm*	*M*24	$K = d_g^{-2.4}\, t_p^{-0.4}\, d_b^{-1.5}$ K=0,000008725
$\theta_r = 15{,}967 \cdot 10^{-9} M - 690{,}76 \cdot 10^{-22} M^3 + 28145{,}78 \cdot 10^{-35} M^5$				$[rad]$
$C_1 = 1.83 \times 10^{-3}$, $C_2 = -1.04 \times 10^{-4}$, $C_3 = 6.38 \times 10^{-6}$				

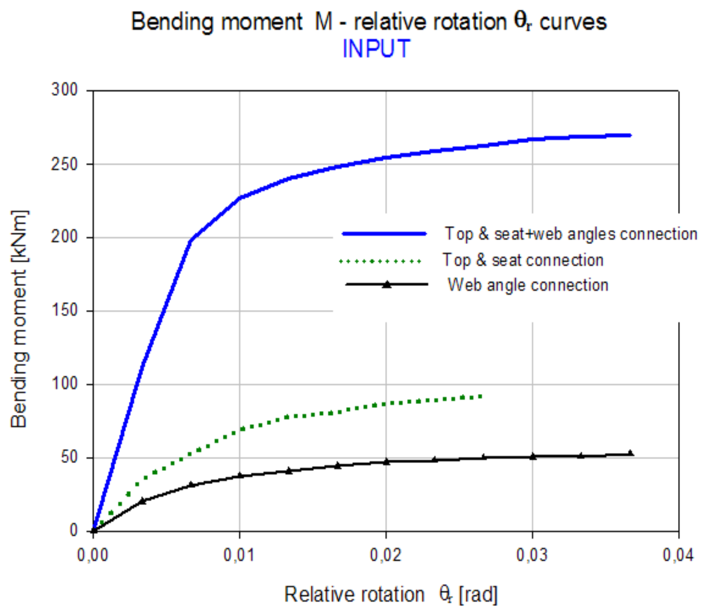


Figure 2. Curves M- θ_r for the studied types of connections

2. Geometrical nonlinear analysis of the semi-rigid steel frames

2.1 Finite element for the bar with elastic connections

The static analysis is formulated through a finite element approach. The FEM model for the bar with elastic connections is represented in Fig. 3. Fundamental vectors associated to the finite element are: nodal displacements $\boldsymbol{q}$, nodal forces $\boldsymbol{Q}$, internal actions $\boldsymbol{X}$ and strain vector $\boldsymbol{x}$.

$$\boldsymbol{Q} = \begin{bmatrix} Q_1 \\ Q_2 \\ Q_3 \\ Q_4 \\ Q_5 \\ Q_6 \end{bmatrix}; \qquad \boldsymbol{q} = \begin{bmatrix} q_1 \\ q_2 \\ q_3 \\ q_4 \\ q_5 \\ q_6 \end{bmatrix}; \qquad \boldsymbol{X} = \begin{bmatrix} M_A \\ M_B \\ N \end{bmatrix}; \qquad \boldsymbol{x} = \begin{bmatrix} \theta_A \\ \theta_B \\ x_3 \end{bmatrix} \tag{2}$$

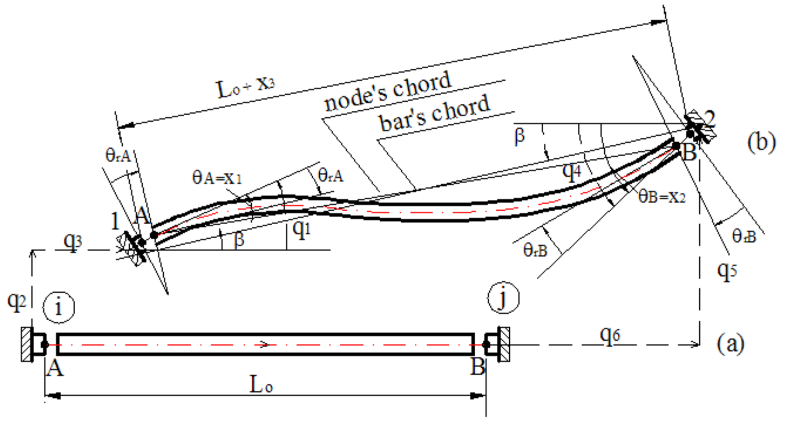


Figure 3. Finite element for the bar with elastic connections at the ends

Equilibrium equations (3) are expressed on the deformed shape of the structure and the boundary conditions are (4):

$$\boldsymbol{Q} = \boldsymbol{A}^T \boldsymbol{X} \tag{3}$$

$$\boldsymbol{x} = \boldsymbol{A}\boldsymbol{q}\,. \tag{4}$$

Matrix $\boldsymbol{A}$ enclose the nonlinear character through the influence of the nodal displacements $\boldsymbol{q}$ upon the strains at the ends of the element $\boldsymbol{x}$ [2]. In addition, it contains also the influence of the finite dimensions of the joints and deviations from the axis [3]:

$$\boldsymbol{A}^T = \begin{bmatrix} 1+\dfrac{a_x}{\overline{L}} & \dfrac{a_x}{\overline{L}} & 0 \\ \dfrac{1}{\overline{L}} & \dfrac{1}{\overline{L}} & -\dfrac{q_4-q_5}{\overline{L}} \\ \dfrac{q_2-q_5+a_x q_1+b_x q_4}{\overline{L}^2} & \dfrac{q_2-q_5+a_x q_1+b_x q_4}{\overline{L}^2} & -\dfrac{L}{\overline{L}} \\ \dfrac{b_x}{\overline{L}} & 1+\dfrac{b_x}{\overline{L}} & 0 \\ -\dfrac{1}{\overline{L}} & -\dfrac{1}{\overline{L}} & -\dfrac{q_4-q_5}{\overline{L}} \\ -\dfrac{q_2-q_5+a_x q_1+b_x q_4}{\overline{L}^2} & -\dfrac{q_2-q_5+a_x q_1+b_x q_4}{\overline{L}^2} & \dfrac{L}{\overline{L}} \end{bmatrix} \tag{5}$$

where a_x, b_x, a_y şi b_y are the eccentricities of nodes 1 and 2 with respect to the connecting sections A şi B and

$$\overline{L} = L_0 + q_6 - q_3 \tag{6}$$

Constitutive law of the material for a bar with elastic connections are:

$$\mathrm{d}\boldsymbol{X} = \boldsymbol{k}\mathrm{d}\boldsymbol{x} \tag{7}$$

Because deformations x_1 and x_2 include the relative rotations θ_{rA} and θ_{rB} due to elastic connections at the ends of the bars it result:

$$\mathrm{d}X_1 = \frac{4EI}{L}(\mathrm{d}x_1 - \mathrm{d}\theta_{rA}) + \frac{2EI}{L}(\mathrm{d}x_2 - \mathrm{d}\theta_{rB})$$
$$\mathrm{d}X_2 = \frac{2EI}{L}(\mathrm{d}x_1 - \mathrm{d}\theta_{rA}) + \frac{2EI}{L}(\mathrm{d}x_2 - \mathrm{d}\theta_{rB})$$
$$\mathrm{d}X_3 = \frac{EA}{L}\mathrm{d}x_3 \tag{8}$$

If R_{iA} and R_{iB} is the initial stiffness at the end of the bar, the static-kinematic duality [4] yields:

$$\mathrm{d}\theta_{rA} = \frac{\mathrm{d}X_1}{R_{iA}} \qquad \mathrm{d}\theta_{rB} = \frac{\mathrm{d}X_2}{R_{iB}} \tag{9}$$

and stiffness matrix of the element is

$$\boldsymbol{k} = \begin{bmatrix} \dfrac{4EIR_{iA}R_{iB}L + 12(EI)^2 R_{iA}}{(R_{iA}L+4EI)(R_{iB}L+4EI)-4(EI)^2} & \dfrac{2EIR_{iA}R_{iB}L}{(R_{iA}L+4EI)(R_{iB}L+4EI)-4(EI)^2} & 0 \\ \dfrac{2EIR_{iA}R_{iB}L}{(R_{iA}L+4EI)(R_{iB}L+4EI)-4(EI)^2} & \dfrac{4EIR_{iA}R_{iB}L + 12(EI)^2 R_{iB}}{(R_{iA}L+4EI)(R_{iB}L+4EI)-4(EI)^2} & 0 \\ 0 & 0 & \dfrac{EA}{L} \end{bmatrix} \tag{10}$$

Accepting a finite formulation, the geometric nonlinear stiffness matrix of the bar with elastic connections is:

$$\boldsymbol{K} = \boldsymbol{A}^T \boldsymbol{k} \boldsymbol{A}\ . \tag{11}$$

2.2 The geometrical nonlinear stiffness matrix

The relationship (11) may be expressed in a particular form by transforming the semi-rigid connections into ideal joints and maintaining the geometrical character of "connection zone with finite dimensions" [5]. It result five cases detailed below.

1) Bar with fixed – semi-rigid ends (Fig. 4)

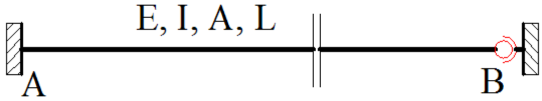


Figure 4. Bar with fixed – semi-rigid ends

Considering $R_{iA} \to \infty$ (the fixed end is at the right end), the stiffness matrix is:

$$\boldsymbol{K}=\begin{bmatrix} \dfrac{4EIR_{iB}L+12(EI)^2}{R_{iB}L^2+4EIL} & \dfrac{2EIR_{iB}L}{R_{iB}L^2+4EIL} & 0 \\ \dfrac{2EIR_{iB}L}{R_{iB}L^2+4EIL} & \dfrac{4EIR_{iB}L}{R_{iB}L^2+4EIL} & 0 \\ 0 & 0 & \dfrac{EA}{L} \end{bmatrix} \tag{12}$$

If the right end is semi-rigid, the condition is $R_{iB}\to\infty$, and the stiffness becomes:

$$\boldsymbol{K}=\begin{bmatrix} \dfrac{4EIR_{iA}L}{R_{iA}L^2+4EIL} & \dfrac{2EIR_{iA}L}{R_{iA}L^2+4EIL} & 0 \\ \dfrac{2EIR_{iA}L}{R_{iA}L^2+4EIL} & \dfrac{4EIR_{iA}L+12(EI)^2}{R_{iA}L^2+4EIL} & 0 \\ 0 & 0 & \dfrac{EA}{L} \end{bmatrix} \tag{13}$$

2) Bar with fixed ends (Fig.5)

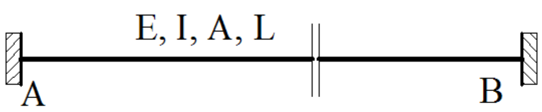


Figure 5. Bar with fixed ends

The conditions imposed in this case are $R_{iA}\to\infty$, $R_{iB}\to\infty$ and results

$$\boldsymbol{K}=\begin{bmatrix} \dfrac{4EI}{L} & \dfrac{2EI}{L} & 0 \\ \dfrac{2EI}{L} & \dfrac{4EI}{L} & 0 \\ 0 & 0 & \dfrac{EA}{L} \end{bmatrix} \tag{14}$$

3) Bar with semi-rigid – hinged ends (Fig. 6)

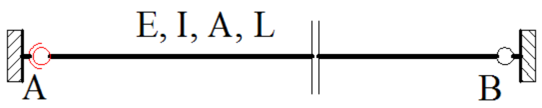


Figure 6. Bar with semi-rigid – fixed ends

In this case $R_{iB}=0$, and the expression for matrix **K** is:

$$\boldsymbol{K} = \begin{bmatrix} \dfrac{3EIR_{iA}}{R_{iA}L + 4EI} & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & \dfrac{EA}{L} \end{bmatrix} \tag{15}$$

If the left end of the bar is semi-rigid, $R_{iA} = 0$, and obtain:

$$\boldsymbol{K} = \begin{bmatrix} 0 & 0 & 0 \\ 0 & \dfrac{3EIR_{iA}}{R_{iA}L + 4EI} & 0 \\ 0 & 0 & \dfrac{EA}{L} \end{bmatrix} \tag{16}$$

4) Bar with fixed – hinged ends (Fig. 7)

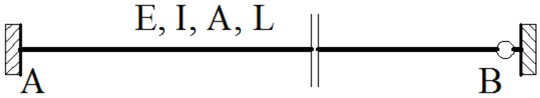


Figure 7. Bar with fixed – hinged ends

The boundary conditions are $R_{iA} \to \infty$ şi $R_{iB} = 0$, and the stiffness matrix is

$$\boldsymbol{K} = \begin{bmatrix} \dfrac{3EI}{L} & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & \dfrac{EA}{L} \end{bmatrix} \tag{17}$$

If the left end is hinged and the right one is fixed, $R_{iA} = 0$ şi $R_{iB} \to \infty$, and result

$$\boldsymbol{K} = \begin{bmatrix} 0 & 0 & 0 \\ 0 & \dfrac{3EI}{L} & 0 \\ 0 & 0 & \dfrac{EA}{L} \end{bmatrix} \tag{18}$$

5) Bar hinged at both ends (Fig. 8)

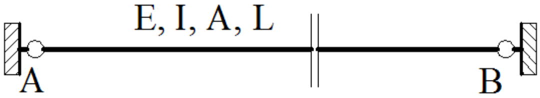


Figure 8. Bar hinged at both ends

The boundary conditions are $R_{iA} = 0$, $R_{iB} = 0$ and the stiffness matrix is

$$\boldsymbol{K} = \begin{bmatrix} 0 & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & \dfrac{EA}{L} \end{bmatrix} \tag{19}$$

3. Numerical results

For theory application, validation of the methodology and comparing the results were chosen structures recognized in the technical literature for highly nonlinear behaviour: William's toggle (Fig. 9) and the diamond shaped frame (Fig. 10). These were analyzed in three variants: (A) with rigid connections at the ends (connection with end plate); (B) with semi-rigid connections at the ends and (C) with semi-rigid connections at the ends and at the apex. The semi-rigid connections in cases (B) and (C) were made in the variants described in Fig. 1 b, c, d.

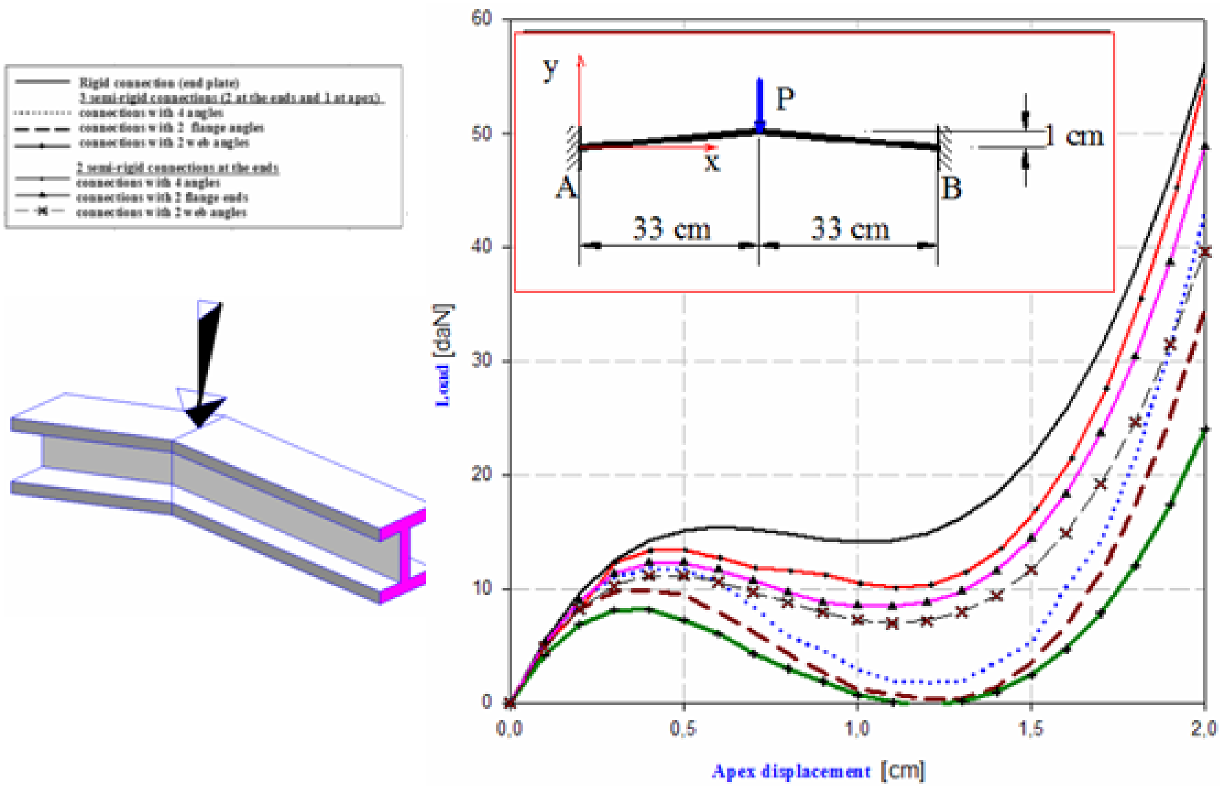


Figure 9. William's toggle

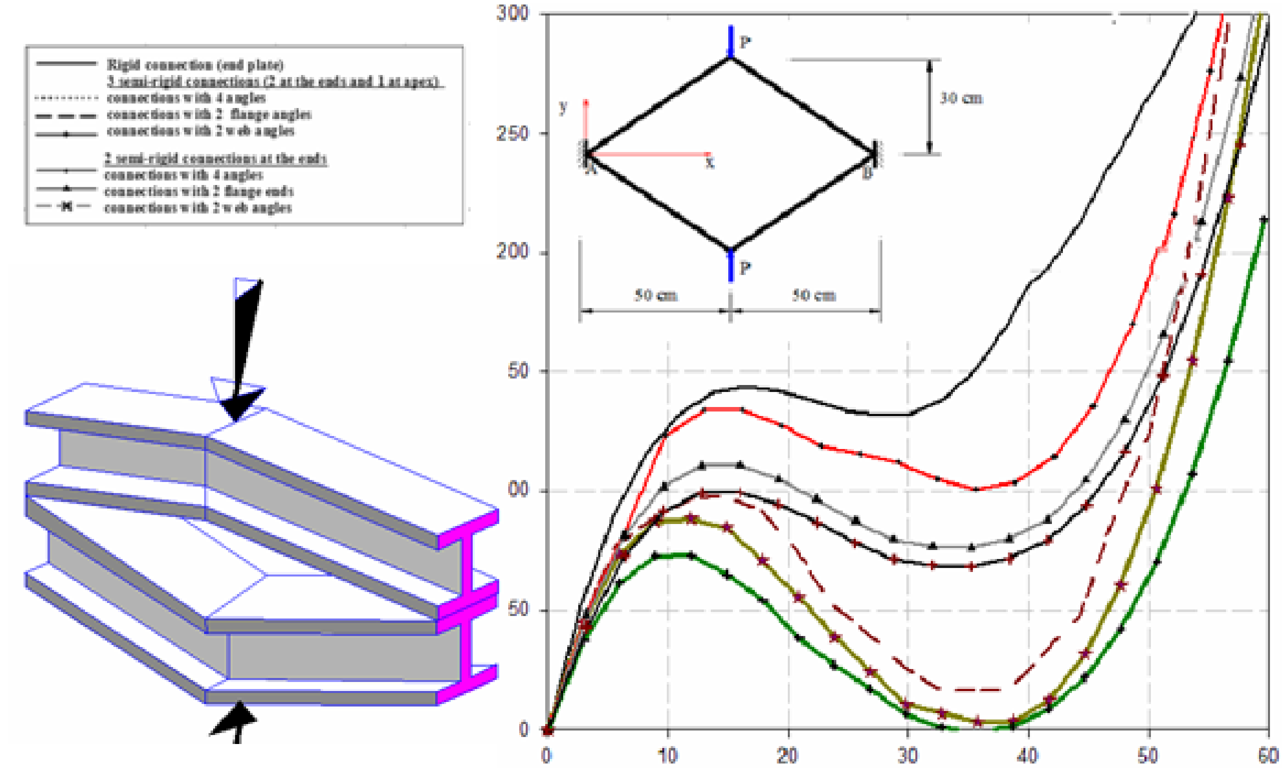


Figure 10. Diamond shaped frame

4. Conclusions

A general conclusion of this study is that semi-rigid steel frames form a separate category of structures which requires treatment by its own procedures and excludes the possibility of assimilation with perfectly rigid or perfectly elastic (hinged) frames and further correlation of the results.

The relationship between the force P and the displacement Δ is strongly influenced by both the type of the connection and the analytical $M-\theta_r$ model. Using the semi-rigid connections described in Fig.1 associated to the Frye and Morris polynomial model, is found, despite significant differences in the final values of the displacement Δ, a similar reaction: a linear and quasi-rigid $P-\Delta$ relationship for low levels of the loading is followed by a strong increase in flexibility for high levels of the loading [6]. A particular case is represented by the connection with 2 web angles (Figure 1, b), polynomial model, which develops a $P-\Delta$ relationship specific for the geometrically nonlinear analysis.

In terms of relationship $P-\Delta$, connections with 2 flanges, respectively 2 web angles, associated to $M-\theta_r$ polynomial model, are very close. Connections with 4 angles (2 web angles and 2 flange angles), associated to the polynomial model are less elastic and may be approximated by a bi-linear $P-\Delta$ variation. Connections with end plate, associated to the polynomial model are practically quasi-rigid.

Numerical experiments show a very similar behaviour – regardless of the structure studied – for the same connection type and analytical model $M-\theta_r$.

Nonlinearity of $P-\Delta$ curves, specific for all semi-rigid structures, suggests pseudo elastic-plastic behaviour, assuming the strain energy dissipation in the joint sections. The ability of strain energy dissipation in connection sections of the semi-rigid steel frames opens, in turn, the possibility of imposing structural ductility and therefore the control of the structural response to seismic actions.

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An analysis of access point density on rural motorways in some EU countries and a proposal for the Transylvania Motorway

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Abstract

The positioning and density of motorway access points are closely related to the mobility (and other) benefits brought to the inhabitants of the areas crossed by the motorway. The present paper analyzes the density of access points from the conventional road network for 19 motorways from six European Union countries, compared to the Transylvania Motorway in Romania. While the mean of the average distances between access points is, for the analyzed motorways, 8.1 km in Austria, 8.6 km in Hungary, 12.5 km in France, 14.3 km in Croatia and 14.5 km in Portugal, according to the current plans the Transylvania Motorway will only be served, on average, by one access point for every 25.5 km.
This paper proposes and argues for improvements on the already planned 20 access points for this motorway, as well as positions for 13 additional access points. Their construction would reduce the average distance between access points on the Transylvania Motorway to 12.8 km – a value which is much closer to values encountered in European regions with population densities similar to Transylvania.

Rezumat

Poziţionarea şi densitatea nodurilor de acces pe autostrăzi sunt în strânsă legătură cu beneficiile de mobilitate şi de alte tipuri aduse de autostradă locuitorilor din zonele traversate de aceasta. În prezenta lucrare a fost analizată densitatea punctelor de acces din reţeaua rutieră convenţională pentru 19 autostrăzi din şase ţări ale Uniunii Europene, comparativ cu Autostrada Transilvania. În vreme ce media distanţelor medii între punctele de acces este, pentru autostrăzile analizate, de 8,1 km în Austria, 8,6 km în Ungaria, 12,5 km în Franţa, 14,3 km în Croaţia şi 14,5 km în Portugalia, conform planurilor actuale pe Autostrada Transilvania ar urma să fie, în medie, un punct de acces la doar fiecare 25,5 km.
Prezenta lucrare propune şi argumentează atât îmbunătăţiri pentru cele 20 de puncte de acces planificate pe această autostradă, cât şi poziţii pentru 13 puncte de acces suplimentare. Construcţia acestora ar reduce valoarea medie a distanţei între punctele de acces pe Autostrada Transilvania la 12,8 km – o valoare mult mai apropiată de cele întâlnite în regiuni europene cu o densitate a populaţiei similară cu cea a Transilvaniei.

Keywords: Urban motorways, interchanges, motorway access points, motorway planning, urban freeways, freeway planning.

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1. Introduction

The positioning and spacing of motorway access points[2] has a great impact on the direct and indirect benefits the motorway brings to the life of inhabitants along the motorway corridor. Furthermore, once designed and built, changes regarding the access points (such as addition, modification or removal) are rather difficult, costly and time consuming. These aspects would imply that during the motorway's design stage great care should be taken regarding the positioning and design of interchanges.

While motorway design standards are in principle harmonized, there is no general consensus regarding the regulations for the distance between access points.

The "Trans-European North-South Motorway Standards" [1] lists the following factors as relevant for the positioning of access points: trip length, size of the urban areas, predicted traffic volumes, cost of interchange construction, congestion control and the possibility of advance signing. However, the document only makes the following ambiguous statement regarding access point spacing: "[t]he distance between two successive interchanges is an element of great importance in ensuring the desired level of service".

In the United States, the AASHTO Interstate Access Guide [2] specifies a minimum distance of 1 mile (1.6 km) between interchanges in urban areas and 3 miles (4.8 km) between interchanges in rural areas. The document states that longer intervals may be needed between "system interchanges" (major interchanges, usually characterized by higher traffic volumes, multi-lane and / or longer ramps) and other interchanges. Nevertheless, some states may have their own regulations: the Caltrans Highway Design Manual specifies that "[t]he minimum interchange spacing shall be 1.5 km in urban areas, 3.0 km in rural areas, and 3.0 km between freeway-to-freeway interchanges and local street interchanges" [3]. The ODOT Highway Design Manual [4] recommends larger intervals between access points: 3 miles (4.8 km) for urban motorways and 6 miles (9.6 km) for rural motorways.

Section 2 of this paper analyzes and discusses interchange spacing on 19 motorways in six European Union countries. Section 3 discusses and proposes improvements regarding the current plans for the Transylvania Motorway – a 416 km motorway in Romania under planning and construction, and currently the largest road infrastructure project in Europe.

2. An analysis of rural motorways interchange density throughout the EU

The purpose of this section is to see how access points are spaced on rural motorways in various countries throughout the European Union. For this purpose, 19 motorway sectors have been selected in six countries other than Romania.

2.1. Selection of the motorways to be analyzed

Since they are intended to work as a comparison for the Transylvania Motorway, an attempt was made to select motorways that are serving areas of similar population density and urban-rural development split. Figure 1 shows a representation of the 19 study motorways and of the Transylvania Motorway on a map showing the population density at NUTS3 level in 2009[3].

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2 Throughout this paper, we use the term "interchange" (or "exit") for motorway access points from the conventional road network and "junction" for motorway access points from other access-controlled facilities (motorways or roads with motorway characteristics).

3 Source: Nordregio (Nordic Centre for Spatial Development). The legend regarding density values is not shown, since only the relative density is relevant.

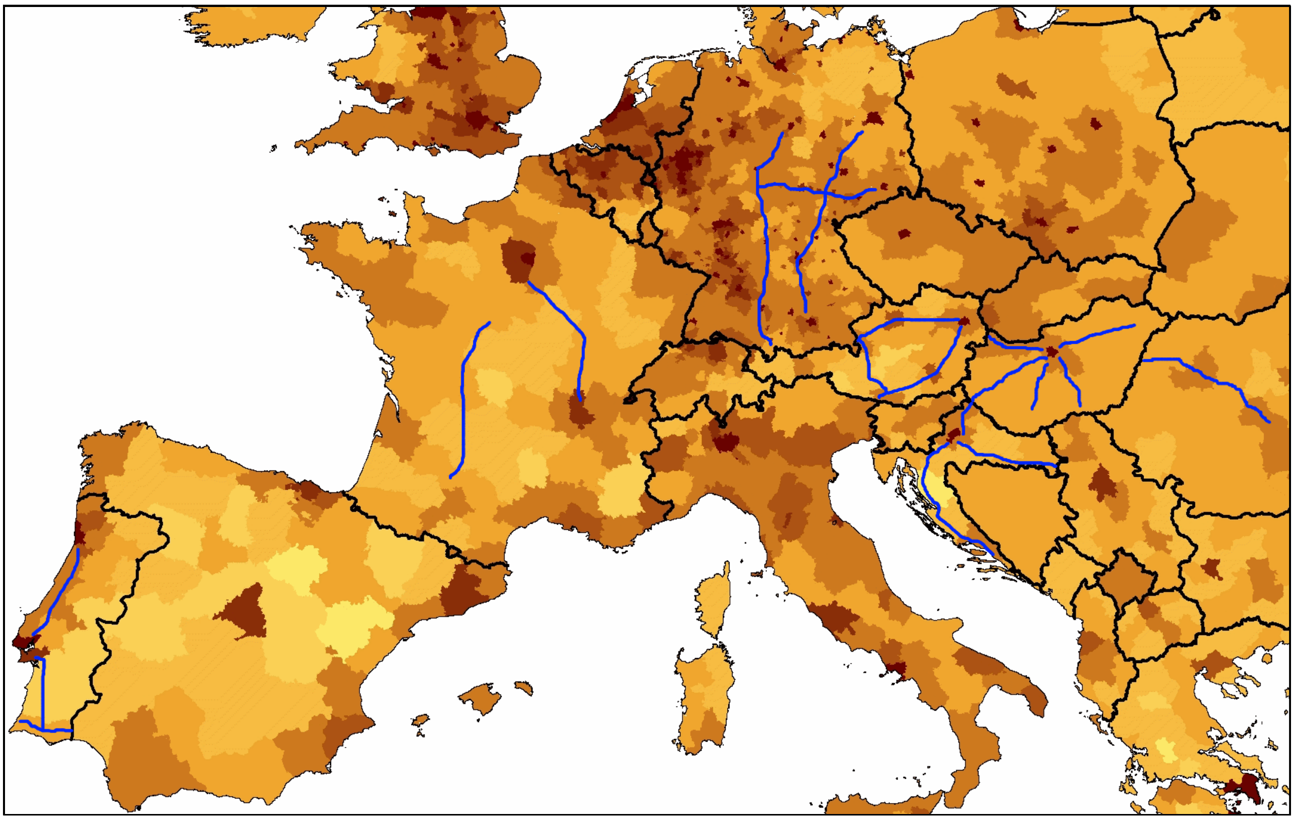


Figure 1. The studied motorway segments in the context of the regional population density

Since another relevant factor is the degree of urban-rural population split, the map in Figure 2 has been used to check for similarities in urban-rural territorial characteristics.

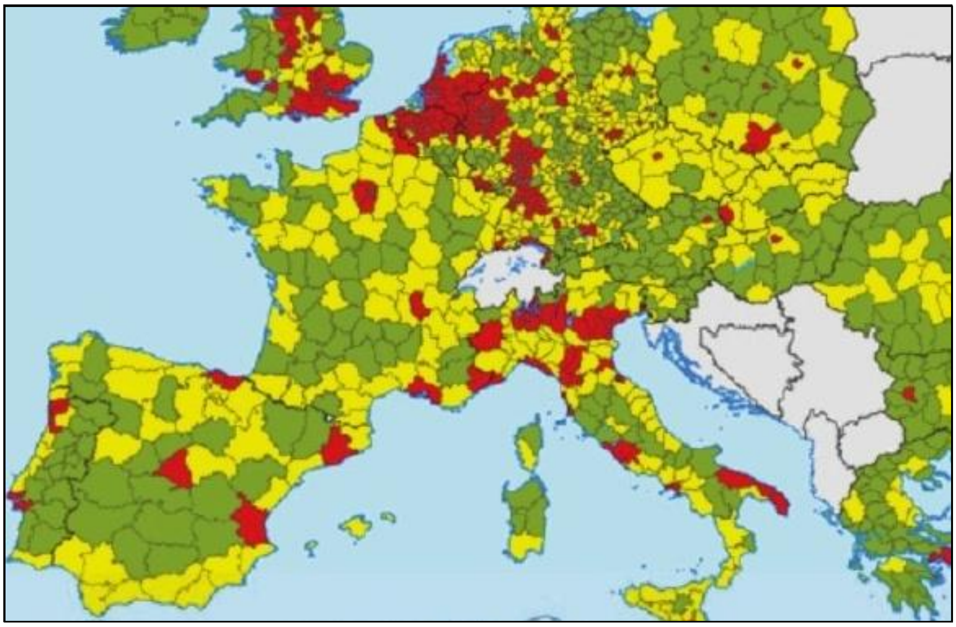


Figure 2. Urban-rural map of the area containing the studied motorway segments[4]
(red: predominantly urban; yellow: significantly rural; green: predominantly rural)

In addition, sections that function as urban or peri-urban motorways have been as much as possible excluded from this study, since the higher access point density encountered on these

4 Source: Eurostat, SIRE-database (E4 unit)

sections would distort data regarding access spacing on rural motorways.

2.2. Methodology of counting the access points

Each motorway sector was selected between two access points from the conventional road network (i.e. neither end was at a junction with other motorways or expressways). For this sector, motorway access points were counted according to the following principles:

i. Only access points from the conventional roads are counted. However, if there are exits on motorways connecting to the motorway sector studied, positioned very close to the motorway-motorway junction, they were either counted, partially counted or not counted at all, according to their distance from the studied motorway (only access points less than 5 km away were considered) and also according to the presence on the main motorway of access points that would duplicate their purpose. Figure 3 shows two cases where access points on branching motorways were counted.

a). Junction of A1 and S33 in Austria.
Although exit 56 is present on the main motorway close to the junction at pk[5] 55, exit 2 on S33 is still counted as a half-exit for A1 since it plays an eastbound A1 access role for an important part of of Sankt Pölten.

b). Junction of A1 and A23 in Portugal (pk 94 on A1). A23's interchange with N3 is counted as a full access point for A1, since the closest access points on A1 are either 20 km north (at pk 114) or 29 km south (at pk 65).

Figure 3. Treatment of access points on motorways connecting to the analyzed motorway sector

ii. Incomplete access points, i.e. where the existing ramps do not serve all four possible directions to/from the motorway, were counted as ¼, ½ or ¾ exits.

iii. "Hidden" interchanges, i.e. access points to the conventional road network that originate from within service areas on the motorway, were counted normally.

iv. Only one of the two access points at the beginning and the end of the analyzed sector was counted, since the average distance between access points on a motorway with a length of L km with n access points is L/(n-1) km/access point.

2.3. Description of the studied motorway sectors

a). Austria

[5] pk = kilometer position.

A1 (West Autobahn) connects Vienna with Sankt Pölten (approximate population - ap: 51,000), Linz (ap: 189,000; greater Linz area: 271,000) which is part of the Linz-Wels-Steyr metropolitan area (460,000) and Salzburg (148,000), after which it ends at the AT/DE border. The analyzed sector is between km 9 (Vienna west exit) and km 297 (last exit in Austria).

A2 (Süd Autobahn) traces the eastern Austrian border, and connects Vienna with Graz (ap: 292,000), Klagenfurt (93,000) and Villach (59,000) and then ends at the AT/IT border. The sector analyzed is between km 47 (the southernmost exit serving Wiener Neustadt, at expressway S4 and national road 17) and km 366 (last exit in Austria). Specifically excluded from the analysis is the Vienna – Wiener Neustadt section, since this serves what is essentially a periurban area of Vienna.

A10 (Tauern Autobahn) crosses the Niedere Tauern mountains, connecting Salzburg (on A1) and Villach (on A2). There are no significant cities (population greater than 50,000) in the A10 corridor. The entire motorway was analyzed, between km 8 (first interchange, south of Salzburg) and km 178 (last interchange before Knotten Villach, where A2, A10 and A11 meet).

b). Hungary

M1 is the major connection between Hungary and Western Europe. Starting from Budapest, it passes near Tatabánya (ap: 76,000) and Győr (130,000; metropolitan area: 183,000), heading towards Vienna. The sector analyzed is between km 22 (first exit after the M1/M0 junction – M0 is the motorway bypass of Budapest) and km 171 (last exit in Hungary).

M3 leads east from Budapest, and serves Eger (ap: 56,000), Miskolc (169,000) and Nyíregyháza (119,000). In the future, the motorway will be extended towards Ukraine and Romania. The sector analyzed is between km 27 (first exit after the M3/M31 junction) and km 234 (where the motorway currently ends, southeast of Nyíregyháza).

M5 connects Budapest with the Serbian border, serving the cities of Kecskemét (ap: 112,000) and Szeged (ap of the urban area: 201,000). The sector analyzed is between km 35 (first exit after the M5/M0 junction) and km 172 (last exit in Hungary).

M6 heads south from Budapest along the Danube. There are no major cities along its corridor. The sector analyzed is between km 22 (which is the last exit serving Érd, essentially a part of the Budapest metropolitan area) and km 191 (last exit before the junction with M60, where the M6 currently ends.

M7 connects Budapest with the Balaton lake area and the Croatian border. It serves the cities Székesfehérvár (ap: 102,000) and Nagykanizsa (50,000). The sector analyzed is between km 18 (first exit after the M7/M0 junction) and km 230 (last exit in Hungary; to be exact, this is the position of the M7/M70 junction, but the first exit on M70, at national road 7, is only about 2 kilometers away).

c). Croatia

A1 starts from Zagreb and stretches along the Croatian Dalmatian coast, with the cities of Zadar (ap: 73,000) and Split (189,000) close to its alignment. The segment studied is between km 14 (first exit after A1's junction with the Zagreb motorway bypass) and 454 (where the motorway currently ends).

A3 heads east from Zagreb towards Belgrade. The only notable city along its route is Slavonski Brod (ap: 65,000). The segment analyzed is between km 49 (first exit after the Zagreb motorway bypass) and km 304 (last exit in Croatia).

A4 starts from Zagreb and connects to motorway M7 in Hungary. There are no cities with a population greater than 50,000 along this motorway. The segment analyzed is between km 3 (first exit in Croatia after the Hungarian border) and km 94 (last interchange before the Zagreb motorway bypass).

d). Portugal

A1 (Auto-estrada do Norte) is the main motorway connecting this country's two largest cities – Lisbon and Porto. It passes along Leiria (ap: 50,000), Coimbra (139,000) and Aveiro

(67,000). The segment analyzed is between km 14 (where A1's junction with CREL – Circular Regional Externa de Lisboa – and also an exit to road N10 are present) and km 291 (last exit before the junction with A20 – Circular Regional Interna do Porto).

A2 (Auto-estrada do Sul) connects Lisbon with the southern part of the country, and although it serves the city of Setubal (ap: 121,000), this motorway is, among all 19 motorways studied, the one that crosses the least populous area. The segment analyzed is between km 24 (junction with A39 and exit for Coina) and km 228 (last exit before A2 ends with its junction with A22).

A22 (Via Infante de Sagres) follows the southern coast of Portugal in the Algarve region. The segment analyzed is between km 0 (where the motorway currently starts, at N120 near Bensafrim) and km 120 (the last exit before the border with Spain).

e). Germany

A4 crosses the central part of the country from west (Aachen) to east (Dresden), linking to the Dutch A76 and the Polish A4. It comprises a western segment of 156 km between the NL/DE border and A45, and an eastern segment of 429 km between A7 and the DE/PL border. The gap between these two segments is reasonably well served by a route mainly comprising segments of A45 and A5, and currently no plans exist to close this gap. The segment analyzed is between km 357 (exit 32, Bad Hersfeld, the first exit east of the junction with A7) and km 19 (exit 77a, Wilsdruff, the last exit before the junction with A17, after which the A4 enters the Dresden urban area, which was deliberately excluded from the study).

A7 crosses the country from north to south, and with a length of 963 km, it is the longest German motorway. The segment chosen for analysis stretches from km 207 (exit 65, first exit south of the A7/A39 interchange) and km 961 (the last exit before the border with Austria). The first 207 kilometers were omitted from the study since they serve the urban areas of Hamburg and Hanover.

A9, another north – south motorway, connects Berlin and Munich. The segment analyzed is between km 2 (exit 2, south of Dreieck Potsdam with the Berlin Ring motorway A10) and km 500 (exit 67, the last exit before the A9/A92 junction, after which the Munich urban area begins).

f). France

A6 (Autoroute du Soleil) connects Paris with Lyon. The segment analyzed is between km 42 (exit 13, Milly-la-Forêt, which is the point where the suburban area southeast of Paris ends) and km 445 (exit 33 Limonest, before the Lyon urban area begins).

A20 (L'Occitane) is part of the Paris – Toulouse north – south axis. The segment analyzed is between km 1 (first exit after the motorway branches off of A71) and km 436 (last exit before the motorway ends in A62).

2.4. Results and discussion

For each of the 19 motorway segments analyzed, exit lists have been obtained, mainly from the website http://motorways-exitlists.com/. Using the application Google Earth, the alignment of each motorway segment has been inspected thoroughly, and the number of access points has been computed in accordance with the criteria presented in section 2.2. Finally, the average distance between access points has been computed by dividing the length of each motorway segment described in the previous section to the number of access points on that segment. The resulting data is presented in Table 1.

Table 1. Average distance between access points on the analyzed motorway segments

Segment number	Country	Motorway	Length (km)	Number of exits	Average distance between exits (km)
1	AT	A1	288	39	7.4
2	AT	A2	322	42	7.7

3	AT	A10	170	18.5	9.2
4	HU	M1	149	20	7.5
5	HU	M3	207	17	12.2
6	HU	M5	137	14	9.8
7	HU	M6	169	23	7.3
8	HU	M7	212	33	6.4
9	HR	A1	440	28	15.7
10	HR	A3	255	14	18.2
11	HR	A4	91	10	9.1
12	PT	A1	277	21	13.2
13	PT	A2	204	9	22.7
14	PT	A22	130	17	7.6
15	DE	A4	307	45.5	6.7
16	DE	A7	599	72	8.3
17	DE	A9	497	34	14.6
18	FR	A6	404	23	17.5
19	FR	A20	435	58	7.5
20	RO	A3	408	16	25.5

The graph in Figure 4 represents this data grouped for each country, with the average distance between access points (ADBAP) represented on the y-axis.

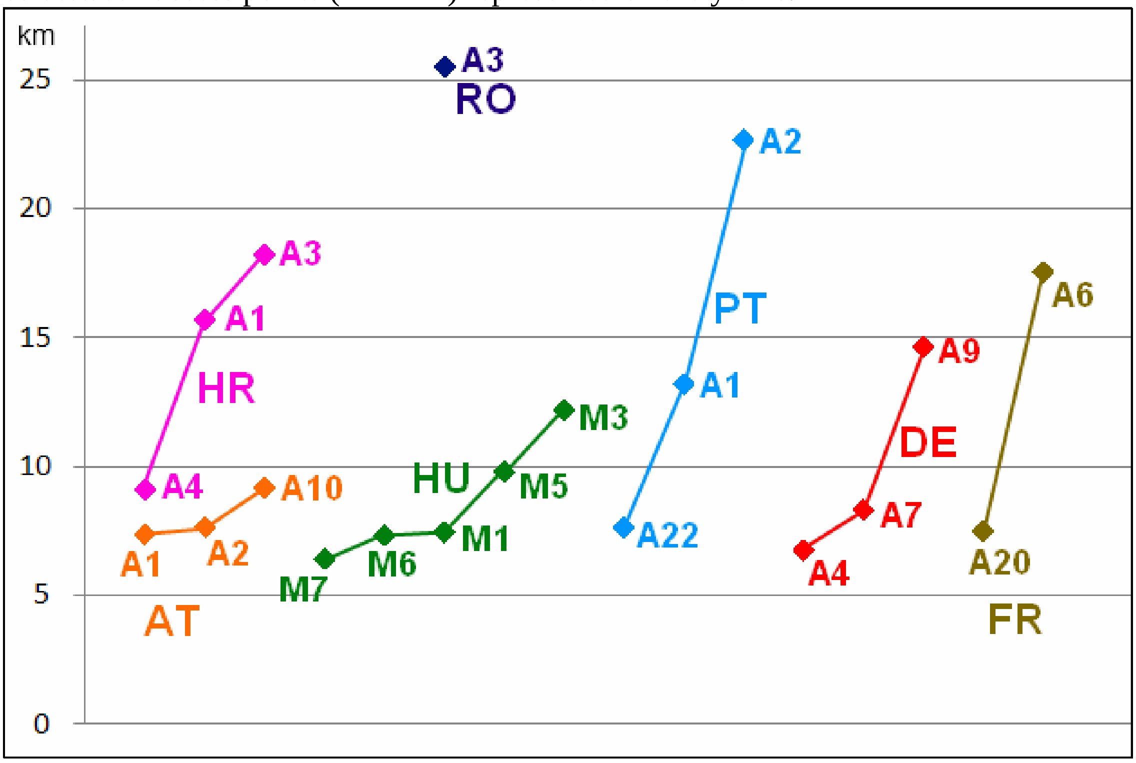


Figure 4. Average distance, in kilometers, between consecutive motorway access points on the twenty motorway segments analyzed

The country with the most frequent motorway access points is Austria (ADBAP = 8.1 km). It is worth noting that A10, a motorway that crosses the Tauern mountains has an ADBAP of 9.2 km – which means that access points are almost three times more frequent on this motorway compared to the Transylvania Motorway, despite the lower population density and more difficult terrain encountered in the Austrian Alps. Hungary has the next lowest average ADBAP, of 8.6 km. In fact, M7 is the motorway with the most frequent access points among all twenty motorway segments, even though the largest city served by it, Székesfehérvár, has a population of only about 100,000. Despite the higher population density and the greater number of cities served, the German motorways average an ADBAP of 9.9 km, however with the access point density on A9 being less than half that of A4. Finally, the three countries that traditionally have built tolled motorways in a closed system have significantly higher average ADBAP values (12.5 for France, 14.3 for Croatia and 14.5 for Portugal). Distance between access points is higher for motorways tolled in a closed system (A1, A3 and A4 in HR, A1 and A2 in PT and A6 in FR), with an average ADBAP = 16.1 km, almost double compared to the other 13 non-Romanian motorways, where the average ADBAP = 8.6 km.

Nevertheless, the Transylvania Motorway has an ADBAP value of 25.5 km, significantly higher than that of any of the 19 other motorway segments. In fact, the only motorway that even comes close to this value is the Portuguese A2, with ADBAP = 22.7 km. However, this motorway is mainly within the Alentejo region, that has a population density of only 24.1 inhabitants/km^2, much lower than the population densities of the Nord-vest (80.9 inhabitants/km^2) and Centru (74.8 inhabitants/km^2) regions crossed by the Transylvania Motorway [5].

3. The author's proposal regarding the interchanges to be built on the Transylvania Motorway

This section presents the author's proposals in regards to the access points that should be constructed on the Transylvania Motorway. The discussion presents issues both related to the currently planned 20 access points (of which 17 are to be constructed now, and 3 at some point in the future) and also proposes 13 additional access points.

Table 2 presents all 33 access points that should be constructed on the Transylvania Motorway, in the author's opinion.

Table 2. Proposed interchanges to be constructed on the Transylvania Motorway
IC = interchange code as used in this paper.
km = approximate kilometer position on the motorway, where km 0 is on DN 1 west of Braşov and km 408 is the last exit, north of Oradea.
Δ = distance from the previous access point, in kilometers.
Status: Planned now (green) = to be constructed at the same time the motorway is built, according to the current plans; Planned later (yellow) = to be constructed at a later (undefined) moment, according to the current plans; Proposed (pink) = author's new proposal. An asterisk indicates an important modification (as proposed by the author) to an already planned interchange's functionality.
Type: Simple = connects only to conventional roads; Mixed = connects both to conventional roads and high speed roads (motorways and expressways); Mixed [Simple] = currently planned as a simple interchange, but the author proposes to be redesigned as a mixed interchange.
Road: Current official name of the conventional road to which the interchange connects (DN = "drum naţional", national road; DJ = "drum judeţean", county road; DC = „drum comunal", communal road).
Destinations served: Major communes, towns or cities served by this interchange. Usually local destinations are listed before long distance ones.

IC	km	Δ	Status	Type	Road	Destinations served
A	0	5	Planned now	Simple	DN 1	Braşov vest, Codlea
B	6	6	Proposed	Mixed	DJ 112 A	Codlea, Feldioara, Braşov Sf. Gheorghe, Bacău, Buzău,

C	37	31	Proposed	Simple	DN 73A	Şercaia, Şinca Veche
D	45	8	Planned now *	Mixed [Simple]	DN 1	Făgăraş Sibiu, Timişoara, Arad, Piteşti
E	62	17	Proposed	Simple	DJ 105A	Şoarş, Cincu, Jibert, Rupea
F	80	18	Planned later	Simple	DJ 106	Brădeni, Iacobeni, Agnita
G	101	21	Planned now	Simple	DN 14	Daneş, Sighişoara Mediaş, Odorheiu Secuiesc, Miercurea Ciuc
H	109	8	Planned later	Simple	DC 24	Dumbrăveni
I	132	23	Proposed	Simple	DJ 142	Suplac, Coroisânmartin Târnăveni, Bălăuşeri
J	139	7	Planned now *	Mixed [Simple]	DJ 151N	Gheorghe Doja, Acăţari Sovata, Iaşi
K	143	4	Planned now	Simple	New road to DN 15	Târgu Mureş Reghin, Bistriţa
L	146	3	Planned later	Simple	New road to DN 15 and DJ 150B	Aeroportul Târgu Mureş Ungheni
M	165	19	Planned now	Simple	DN 14A	Iernut, Cucerdea Târnăveni, Mediaş
N	179	14	Proposed	Simple	DJ 107G	Luduş, Aţintiş
O	182	3	Planned now	Simple	DN 15	Luduş, Cheţani
P	195	13	Built	Simple	DN 15	Câmpia Turzii, Luna
Q	204	9	Built *	Mixed [Simple]	DN 1	Turda Alba Iulia, Sibiu, Timişoara
R	217	13	Proposed	Mixed [Simple]	New road to DN 1 and DJ 107L	Turda, Petreştii de Jos Cluj-Napoca, Dej
S	229	12	Proposed	Simple	DJ 107R	Ciurila, Cluj-Napoca
T	238	9	Proposed	Simple	DJ 107M	Săvădisla, Băişoara
U	247	9	Built *	Mixed [Simple]	DN 1	Gilău, Cluj-Napoca
V	256	9	Planned now	Simple	DN 1F, DJ 108G	Nădăşel, Cluj-Napoca
W	269	13	Proposed	Simple	DN 1F	Mihăieşti, Topa Mică
X	285	16	Planned now	Simple	DN 1G	Zimbor
Y	301	16	Proposed	Simple	DJ 108A	Românaşi, Agrij Jibou, Ciucea
Z	313	12	Planned now *	Mixed [Simple]	DJ 191C	Zalău Baia Mare, Satu Mare, Halmeu
AA	325	12	Proposed	Simple	DJ 108G	Crasna, Vârşolţ, Şimleu Silvaniei
BB	339	14	Planned now *	Simple	DJ 110E	Nuşfalău, Şimleu Silvaniei, Aleşd

CC	349	10	Proposed	Simple	DJ 108P	Ip, Tăşnad, Carei
DD	358	9	Proposed	Simple	DJ 191B	Suplacu de Barcău
EE	375	17	Planned now	Simple	DJ 191A	Marghita, Valea lui Mihai, Tăşnad
FF	393	18	Planned now *	Simple	DJ 767A	Ciuhoi, Sârbi, Sălard, Săcuieni
GG	407	14	Planned now *	Mixed [Simple]	DN 19	Biharia, Oradea, Satu Mare Arad, Timişoara, Belgrad

In addition to the data presented in the table above, the reader should consult the eight maps provided in Appendix 1, in order to better follow the discussion below.

Interchange A at pk 0, on DN 1 between Codlea and Braşov, should be constructed as planned. Note that the value $\Delta = 5$ recorded here refers to the distance from the last interchange on the Bucharest – Braşov motorway (at DJ 112B, west of Cristian).

A new interchange (B) is proposed to be constructed at km 6. Besides connecting locally to DJ 112A to serve Codlea and Hălchiu, this should be the origin point of a west-east 18.5 km feeder road (constructed also as a motorway) crossing DN 13, DJ 103, DJ 112A, DJ 112 and ending east of Hărman, near the DN 10 / DN 11 intersection. The proposed alignment is shown in dark blue in Appendix 1. This link road is extremely important because it connects to the four national roads converging east of Braşov: DN 10 leading to Buzău and Brăila, DN 2D leading to Focşani and Galaţi, DN 11 leading to Bacău and DN 12 leading to Sfântu Gheorghe and Miercurea Ciuc. Traffic from all these roads will be quickly connected to the Transylvania Motorway, and then be distributed either northwest (on the Transylvania Motorway towards Târgu Mureş, Cluj-Napoca and Oradea), west (via the Făgăraş – Sibiu expressway towards Arad and Timişoara) or south (via the Bucharest – Braşov motorway torwards Ploieşti and Bucharest). Interchange B could be constructed as a cloverleaf with collector-distributor lanes on the Transylvania Motorway.

A newly-proposed interchange C would use DN 73A to serve the communes Şercaia and Şinca Veche, and other settlements in the DN 73A / DN 1 intersection area, but also traffic from DJ 104 leading northeast. Although this interchange is represented as a single unit on DN 73A, it could be conceived as a split interchange, with westbound-connecting ramps positioned on DN 73A between Şercaia and Vad, and eastbound-connecting ramps positioned on DN 1 between Şercaia and Perşani.

Interchange D, on DN 1 east of Făgăraş, should be reconfigured from a simple interchange to a mixed-use interchange, to allow for the future Făgăraş – Sibiu expressway. A cloverleaf with collector-distributor ramps on the Transylvania motorway would also probably be the best choice here, maybe with a directional ramp for the Braşov – Sibiu direction.

A new interchange E is proposed to be constructed east of Şoarş, since this is the intersection point of DJ 104D and DJ 105A, serving not only the neighboring settlements but also the town Rupea.

Interchanges F (Netuş), G (Daneş) and H (Hoghilag) are currently part of the plans, but the first and the third are to be constructed at an undetermined future date. The author believes that their construction should not be delayed.

Interchange I, proposed by the author to be built on DJ 142 near Suplac is also important, since it will serve all the settlements in the Târnava Mică valley. Furthermore, until the Târgu Mureş – Iaşi motorway will be built, this interchange will serve as the access point to the Transylvania Motorway for the traffic originating from northeastern Romania, using DN 13B and DN 13A (Gheorgheni – Praid – Sovata – Bălăuşeri).

Interchange J should serve both as a junction with the future Târgu Mureş – Iaşi motorway, but also as a local access to DJ 151D, near Gheorghe Doja. This would be closely followed by interchange K north of Leordeni, where a linkroad to DN 15 (near Cristeşti) is planned to be constructed. Finally, interchange L is proposed to be built south of DN 15 and west of DJ 151B, mainly to serve Târgu Mureş airport. A linkroad would be needed here too, proposed to originate in the roundabout close to the airport and to end in DJ 151B between Ungheni and Cerghizel.

Nevertheless, the author insists on presenting another proposal for this area, consisting in a complete rerouting of the Transylvania Motorway, as shown in Figure 5. The main benefit of this proposal is the shortening by about 10 km of the Transylvania Motorway alignment. In addition, this would eliminate the succession of the rather closely-spaced interchanges J, K and L (according to the current plans, the distance from J to K is 4 km and the distance from K to L is 3 km).

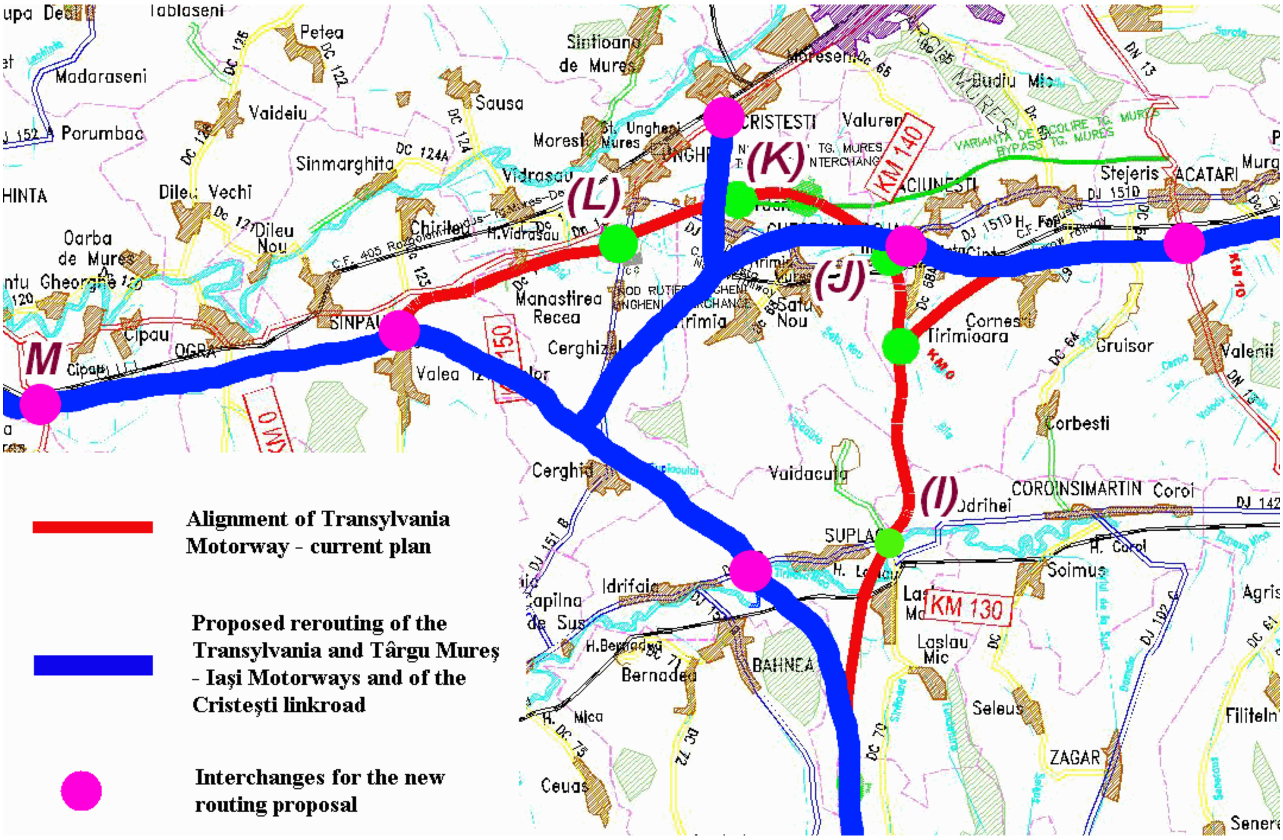


Figure 5. Proposal for a rerouting of the Transylvania Motorway,
Târgu Mureş – Iaşi Motorway, and the link road to Cristeşti

The linkroad to Cristeşti should also be constructed with motorway characteristics, since in the future this could be extended as a motorway or expressway torwards Reghin (and further up north to Bistriţa).

Between interchanges M and O, to be built as planned, the author proposes the addition of another interchange on DJ 107G, between Luduş and Aţintiş. Interchange P, already built, serves Câmpia Turzii and Luna. Interchange Q should serve not only as a local connection to Turda, but also as a junction with the future Turda – Sebeş expressway or motorway. Unfortunately this interchange has already been built according to the original plans, as a double-trumpet, without taking into account the future expressway. However, this could be rather easily modified into a triple trumpet (potentially with directional ramps for the west – south motorway movements) to serve the purposes described herein.

The already-built motorway section between Turda and Gilău should be completed with three additional interchanges:

- Interchange R, on DJ 107L, serving not only the north part of Turda, Petreştii de Jos and the Cheile Turzii nature reserve, but also as a junction with a linkroad to DN 1 (between Mărtineşti and Tureni), which could be later extended as an access motorway towards Cluj-Napoca.

- Interchange S, on DJ 107R near Ciurila, serving also the southern part of Cluj-Napoca;

- Interchange T on DJ 107M, serving Săvădisla and the Băişoara tourist area.

The already-built interchange U near Gilău will have to be modified when the future Cluj-Napoca Urban Motorway will be built.

Between Cluj-Napoca and Zalău, in addition to the planned interchanges V (near Nădăşel) and X (near Zimbor), two more interchanges are proposed: interchange W on DN 1F between Mihăieşti and Topa Mică, serving also local traffic converging on DJ 170B[6] to DJ 109 and DJ 105T, and interchange Y near Românaşi on DJ 108A, serving all settlements on the road axis between Jibou and Ciucea.

Interchange Z near Zalău should be modified as to allow for the construction of the junction of the future Zalău – Ardusat – Baia Mare / Satu Mare expressway ("The Northern Road")[7].

The construction of an interchange (AA) north of Crasna is also important, since five county roads converge here: DJ 108G from north, DJ 191C from east, DJ 108G from south, DJ 191E from southwest and DJ 191C from west. The Nuşfalău interchange (BB) should probably be positioned not on DN 1H, but rather on DJ 110E, since traffic collected from the settlements south of Nuşfalău on DJ 110E and DJ 191H would be significantly larger than that from DN 1H. In any case, a linkroad parallel to the motorway should be constructed between DN 1H, DJ 110E and DJ 191H, to allow traffic from all these three directions to access the motorway without entering Nuşfalău.

Two new interchanges (CC and DD) are proposed, on DJ 109P near Ip (this road connects with the towns Tăşnad and Carei in Satu Mare county) and on DJ 191B near Suplacu de Barcău.

Interchange EE near Marghita and Chiribiş should be built as planned, but interchange FF should not be built on the road connecting Sălard and Sîrbi, but on DJ 767A between Ciuhoi and Sîrbi, since this county road also extends north to the town Săcuieni.

Finally, the last interchange (FF) on DN 19 near Biharia should be modified to allow for the construction of the future Arad – Oradea expressway.

4. Conclusion

The absence of clear standards regarding the location choice for motorway acces points has lead to great variations on this matter within the European Union, and even within individual countries. Decisions regarding this issue are in some cases taken by design companies performing the feasibility studies using "in house empirical methods", without employing a set of complex criteria to reach the optimal set of motorway access points.

The functionality of Transylvania Motorway could be greatly improved by reconsidering the position and design of some of its interchanges, as well as by building 13 new interchanges. The current plan includes 17 interchanges, with 3 more planned to be built in an undefined future. The author of this article believes that all 20 interchanges should be constructed immediately (during the following years, when the motorway will be completed) and that, furthermore, another 13 new interchanges should be designed and built along the motorway construction.

This would essentially halve the average distance between access points (ADBAP) for the Transylvania Motorway from 25.5 km (a value significantly higher than all other EU rural motorways that were analyzed herein) to 12.8 km – a much reasonable value, considering the population density of the regions crossed by this motorway.

Furthermore, this article suggests the redesign of some of the interchanges as to conveniently provide for the future junctions with other planned or suggested motorways and expressways: a northern motorway bypass of Braşov, the Făgăraş – Sibiu expressway, the Târgu Mureş – Iaşi motorway, the Turda – Sebeş expressway, a south – north access motorway to Cluj-Napoca originating near Tureni, the Cluj-Napoca Urban Motorway, the Zalău – Ardusat – Baia

[6] DJ 170B currently bears an anti-systematic number (as it has no connection whatsoever to DN 17), and should hence be renumbered, e.g. as DJ 109W.

[7] Pending on a final decision regarding the alignment of this expressway, the actual junction between Transylvania Motorway and "The Northern Road" could be shifted west of interchange Z (as shown in the map presented in Appendix 1).

Mare / Satu Mare expressway and the Oradea – Arad expressway. Although the construction of these motorways and expressways might not take place for some time, it would be good practice in long-term strategic thinking to provide for these junctions.

Another issue to consider is related to the fact that the Transylvania Motorway interchanges were designed as appropriate for closed-system tolling. Specifically, they are configured as trumpets or double-trumpets, such as to allow the convergence of all traffic flows in one point, in order to be able to install one single toll gate. However, considering that:

a). Electronic tolling is a much more feasible option today than it was years ago, and also much cheaper from an operational point of view;

b). Trumpet and double-trumpet interchanges are more expensive to build and almost always involve a longer travel distance for drivers getting on and off the motorway;

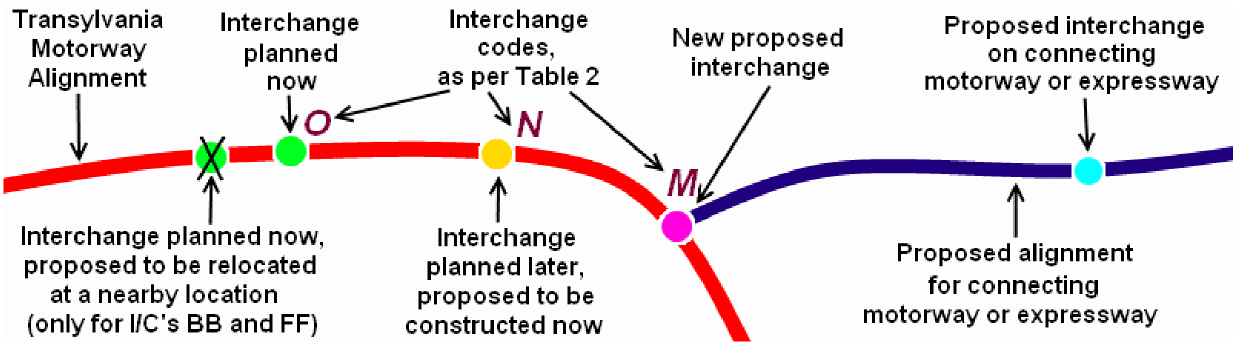
it would be a good idea to redesign and build these interchanges without the "classical" closed-system tolling constraint in mind. If in the future a decision will be made to toll this motorway, this could be easily achieved by e-tolling (similar to, say, the Go-Box system used in Austria for heavy vehicles).

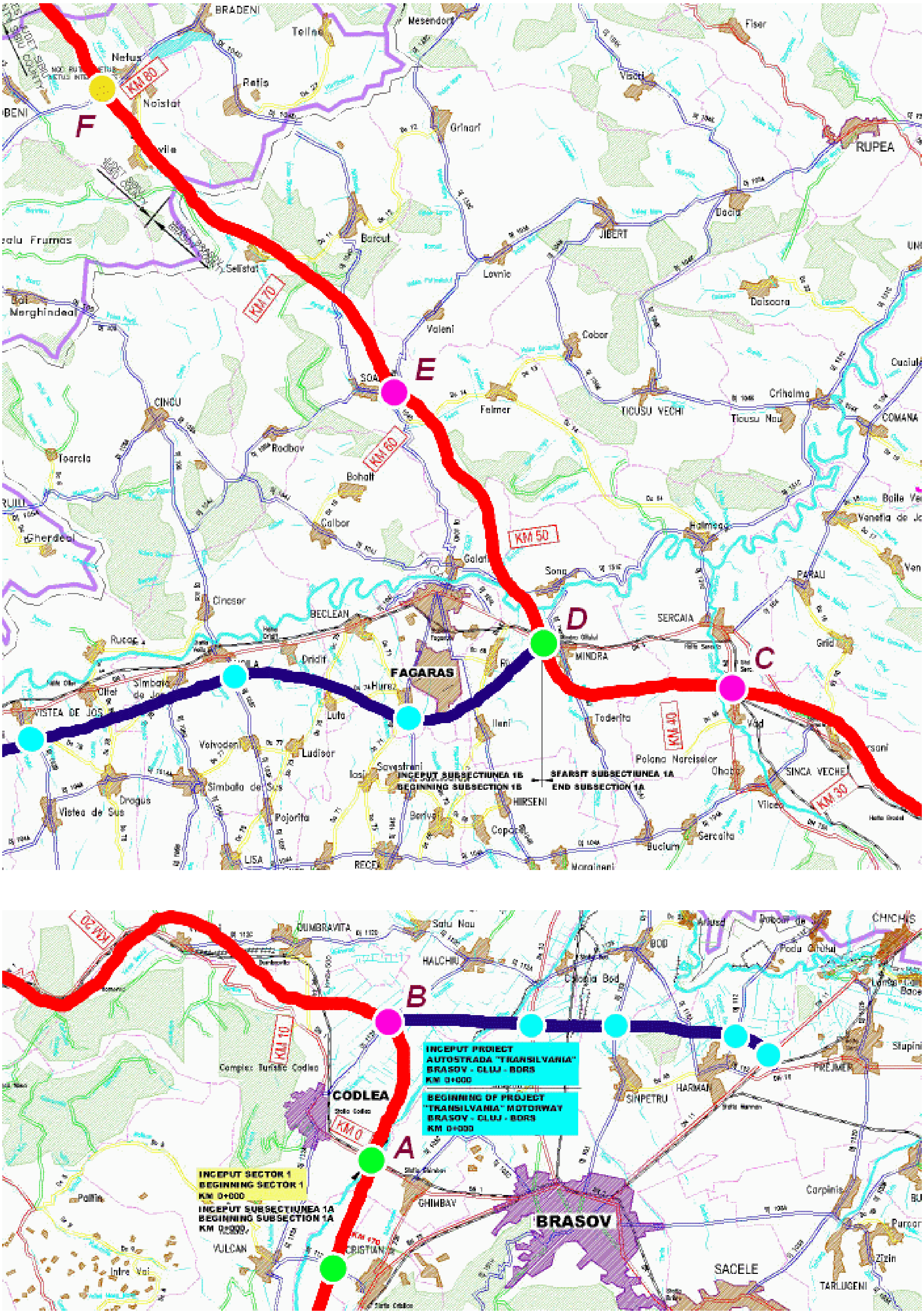
The savings obtained from redesigning and building the currently planned interchanges as diamonds, partial cloverleafs etc. as well as the savings obtained from rerouting the motorway near Târgu Mureş (by eliminating an unneeded 10 km detour) would be more than sufficient to cover the costs of the 13 new proposed interchanges, as well as the cost of building modified versions of the interchanges where needed to provide for junctions with future motorways and expressways.

An idea for future work related to the topic of this paper is to develop a formal mathematical model that would be used to assess the mobility (and other types of) benefits of a motorway interchange. This model could then be used when elaborating feasibility studies for other future motorways, in order to conduct a much more rigorous analysis regarding the positioning and frequency of motorway access points.

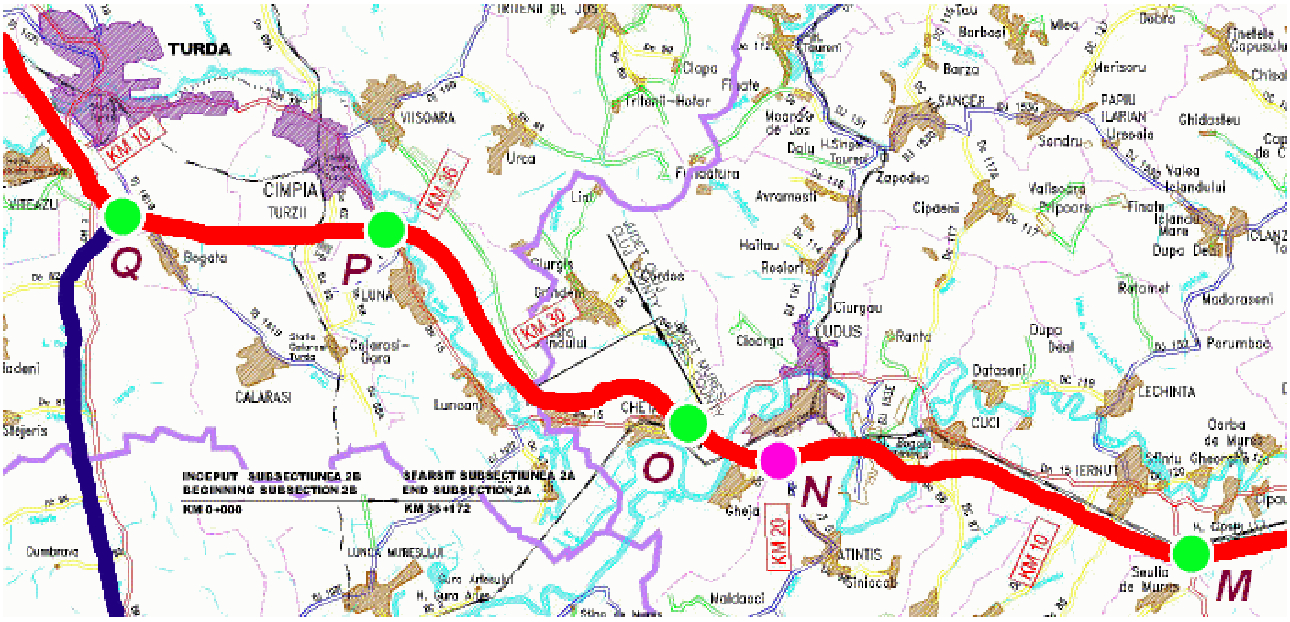
4. Appendix

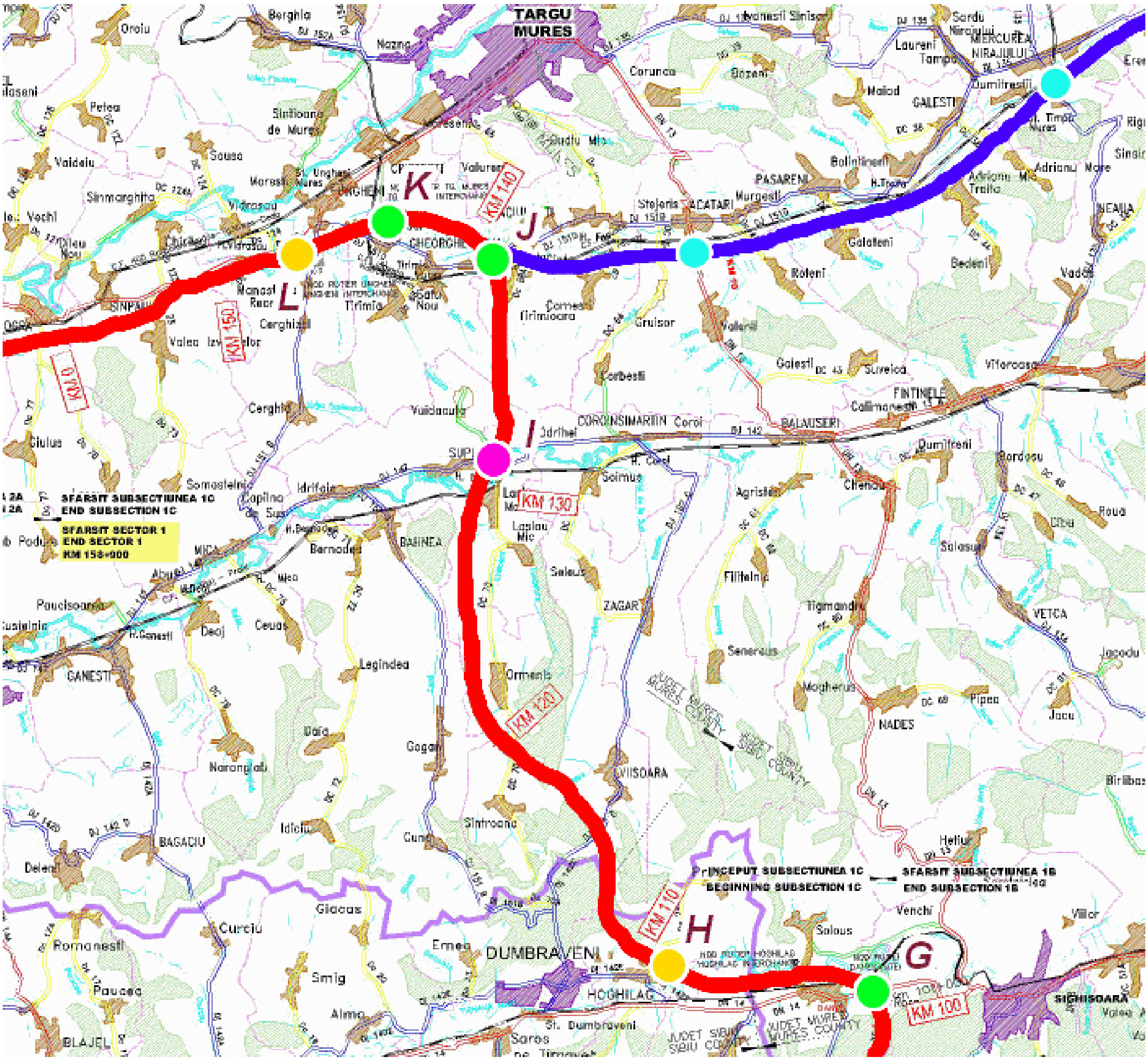
The following four pages comprise eight maps (ordered from east to west but on the same page from north to south) showing the author's proposal for the interchanges on Transylvania Motorway. The following legend is used:



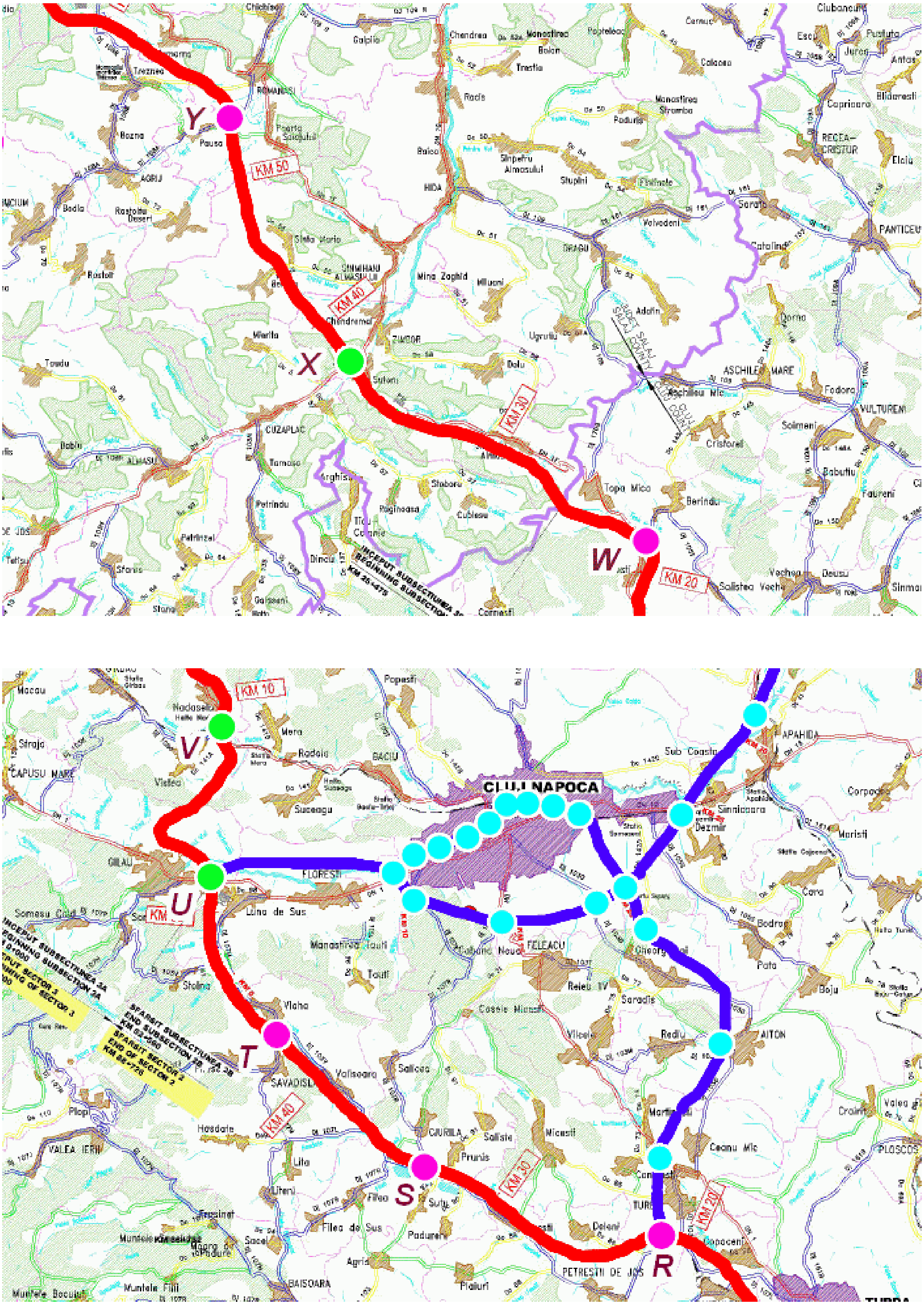


Appendix 1 – page 1. Proposed interchanges on the Transylvania Motorway

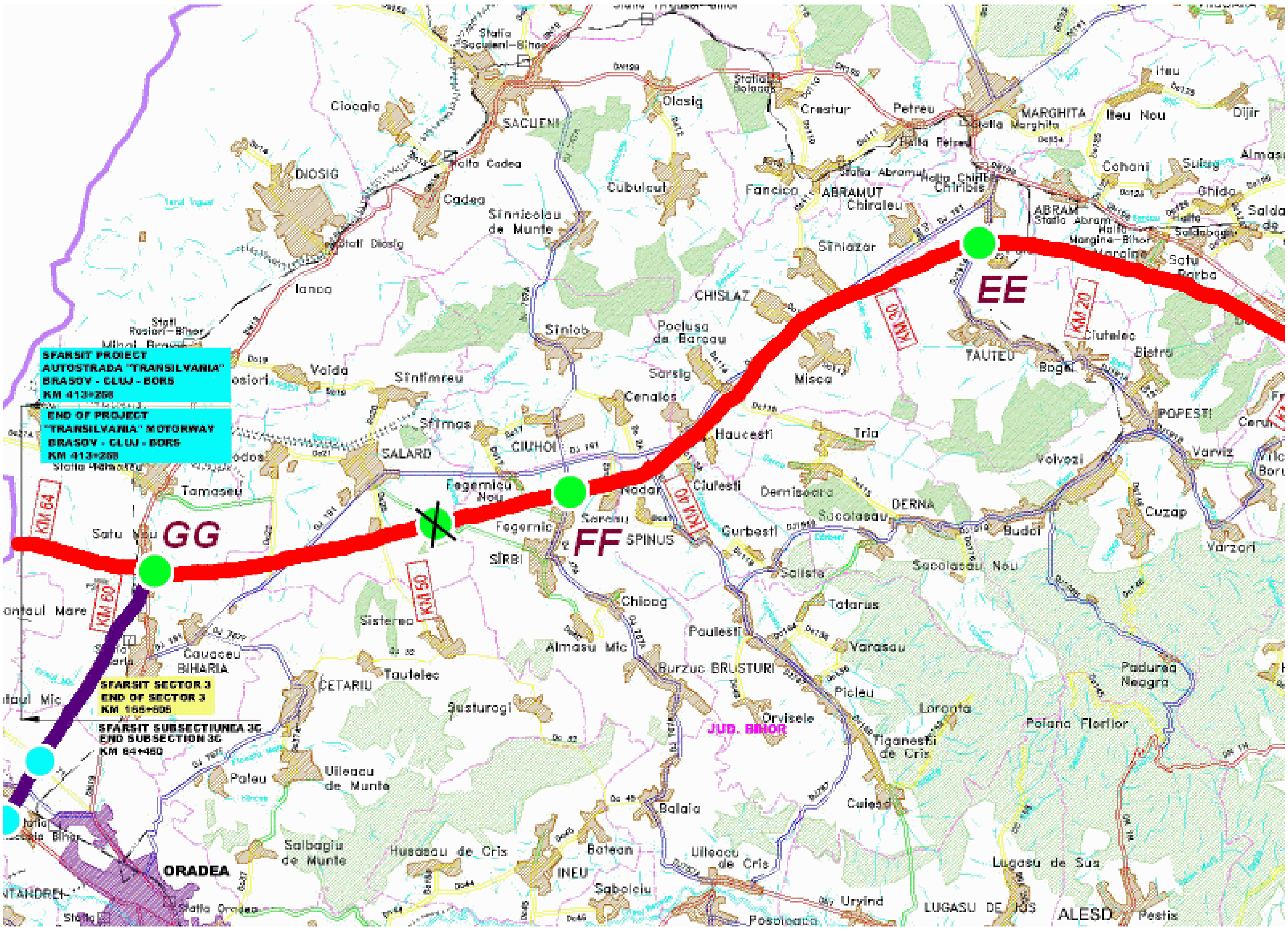


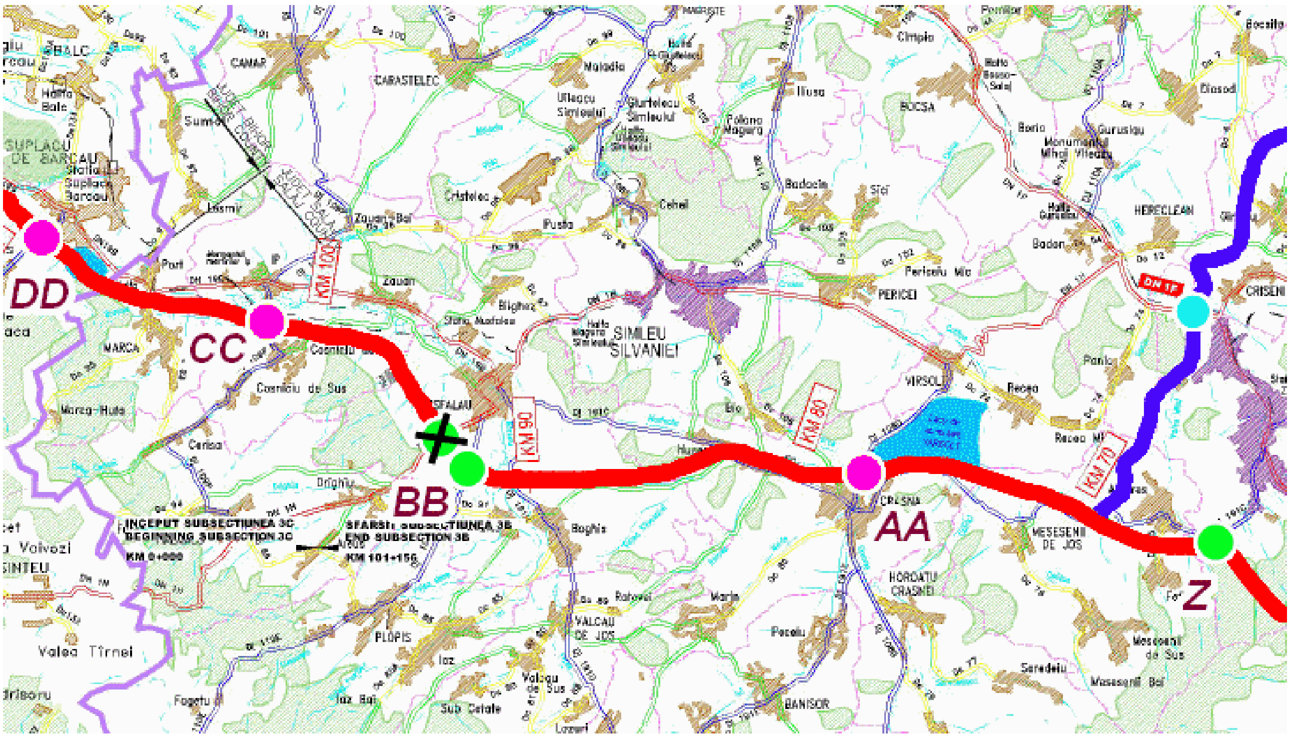


Appendix 1 – page 2. Proposed interchanges on the Transylvania Motorway



Appendix 1 – page 3. Proposed interchanges on the Transylvania Motorway





Appendix 1 – page 4. Proposed interchanges on the Transylvania Motorway

Acknowledgements

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Fire Structural Analysis According to European Codes

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Abstract

This study describes the way in which the prescriptions for structural fire design are presented in the "Eurocodes". In the first chapter the notion of fire safety according to European codes *is described. There are also a few examples of protections to fire. Some advanced structural fire resistance models are shown together with some remarks about their validation procedures. Here, a review of recent advances on this subject is done. In the second chapter a short presentation of the "parts" from the "Eurocodes" which contain the fire design and an explanation of the format applied in these codes in order to obtain the required level of safety is made. Chapter three contains definition of the structural fire resistance followed by examples of some fire tests conducted at Cardington in the British Steel Programme. In chapter four there is an overview of the fire design - the actions and their grouping are presented; the design values for material properties and last the categorization of the methods used for structural fire design is shown for each type of material - structure. The last chapter contains the conclusions of this study.*

Rezumat

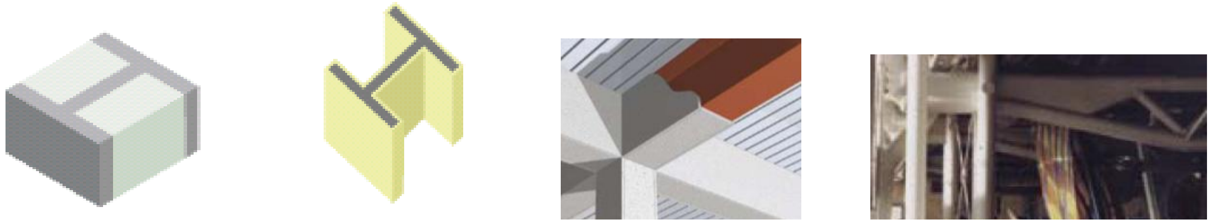
Aceasta lucrare descrie modul in care este prevazuta analiza structurilor supuse la actiuni de tip foc in "eurocoduri". In primul capitol este prezentata notiunea de siguranta la foc prevazuta in normele europene. Sunt aratate si cateva exemple de protectii la foc. Tot in acest capitol se face o introducere in modele avansate de calcul – programe specializate si validarea acestora. Capitolul continua cu o trecere in revista a literaturii recente despre obiectul tratat de aceasta lucrare cu o scurta prezentare a concluziilor acestor studii. In al doilea capitol este realizata o scurta prezentare a partilor din "Eurocoduri" care contin analiza structurilor la foc impreuna cu explicatii referitoare la baza calculului. Capitolul trei contine definitia rezistentei la foc si exemple de teste realizate la Cardington pe structuri cu dimensiuni reale. Capitolul patru prezinta o trecere in revista a calculului la foc si anume: actiunile si gruparea lor; valorile de calcul ale proprietatilor materialelor iar apoi sunt prezentate procedurile de proiectare care pot fi folosite. In ultimul capitol se arata concluziile acestui articol.

Keywords: fire structural analysis, fire safety, fire resistance, fire test, thermal actions, safety factor, reduction factor.

1. Recent Advances on Fire Design

The notion of safety of the structures in case of fire refers to the possibility of a construction to resist fire without collapsing for a period of time. The load bearing capacity of each structural element decreases during fire but following a suitable structural fire design, the civil engineer provides the structure the capacity to resist to fire action for an appropriate period without loss of stability. The majority of the regulations worldwide concerning the fire safety of the buildings are about the safety of the persons inside the building, firemen and also about protecting some important installations inside the building. Example of these provisions: use of fire detecting, sprinklers, smoke dispersers etc. (an installations point of view). These are active measures which prevent the spreading of smoke and fire inside the building.

Other types of specifications are about dimensions of rooms, halls, stairs, doors etc. – from an architectural point of view and about structural engineering measures that avoid the collapse of the beams, columns or slabs. They are known as passive protection to fire. Examples of protections for structural elements are presented in figure 1: a steel column protected by infilling with concrete (figure 1.a); a protected column (figure 1.b.); fibre board protection applied to beams and columns (figure 1.c.); thin film intumescent coating (figure 1.d.) [1].



a) b) c) d)

Fig. 1 - Examples

Frequently used computer applications for numerical calculations are: SAFIR, VULCAN, ABAQUS, COMPSL, STELA, THELMA etc. These use advanced structural fire resistance models. In the following, an example of a slab supported by an open web steel truss is shown; by using an appropriate application one may find the internal forces and deflections of the slab under thermal load and dead-weight (figure 2), [11].

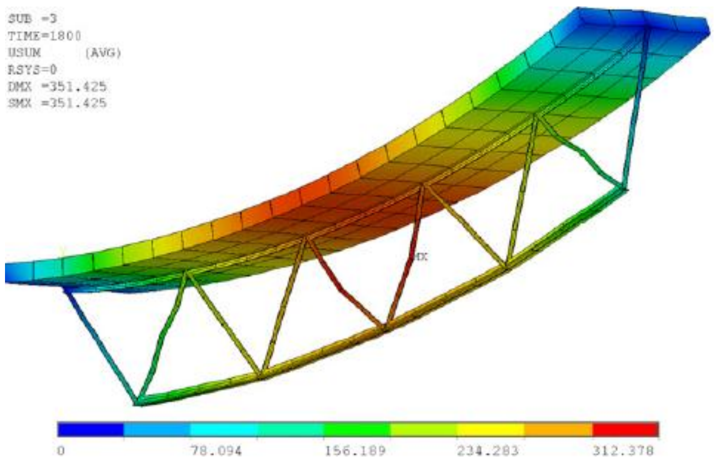


Figure 2. Deflections under thermal load and dead weight [11]

The study of professor Dubina and Zaharia, [12] uses the advanced calculation model SAFIR. In this paper it is stated that "computer programs which use such advanced calculation models must fulfill certain conditions imposed by the Eurocodes and must be validated through relevant test results". In this case the authors used the results of the fire tests realized by ARBED RECHERCHES for columns and beams having composite steel – concrete sections. A conclusion of their study was the fact that SAFIR respects the principles stated by Eurocodes to be considered an advanced calculation model for this type of analysis. The study of Souza Junior, Creus, [13] uses also calculation model SAFIR to realize "a simplified analysis of three dimensional frames under elevated temperature". Numerical investigations were also made by using VULCAN calculation model and FPRCBC according to the prescriptions of Eurocode: Foster, Chladna, [15].

Other authors like Dong and Prasad, [14] made an analysis of the "performance of structural frames under fire loading" by experimental means: the paper describes the experimental results of a furnace

test conducted on three full-scale composite frames. In some articles like Dwaikat, Kodur [16] the authors uses self made software which is then confronted with SAFIR and with test results.
A review of the literature shows that different authors worldwide treated this subject. Many studies were made according Eurocodes with self made computer applications or with the above mentioned applications, they confronted the results with experimental tests made by different organizations using furnaces. Some important studies in this area are described in many papers: Zehfuss, [19] a parametrical fire model called "iBMB parametric fire curve" is presented; Gillie, [20] „identifies the key phenomena needed for accurate high temperature structural modelling"; Jianyong [21], contains an analysis of "computer integration technology"; Iqbal, [22] studies the thermo-mechanical performance of steel columns; Chung, [23] a calculation method using "the theory of volume averaging" for computing the stresses; Xu [24], the authors made a computer application called „RCSSCF" to calculate the columns and they compare the results with tests made on four full-scale columns; Alderighi, [25] a "numerical investigation" of the earthquake resistant composite structures was made; Di Capua [26] makes a thermal analysis that than is compared to experimental results. In the study, the properties of steel and concrete are according to Eurocodes 1 and 2 part 1.2. In some important books concerning the issue like Moore, Cooke [17], full scale fire tests are discussed and then confronted with fire – programmes. Another important book is of professor Zaharia, Franssen, Kodur [18], which „provides guidance for those wishing to apply engineering methodologies for fire design of steel structures" with examples on how a steel structure may be designed.

2. Description of Parts 1.2. of the Eurocodes

Part 1.2. of the Eurocodes refers to "Structural fire design". Any structure designed according to part 1.1. and fulfilling the supplementary requirements of part 1.2. should have the required fire resistance.
In the following it is presented each Eurocode with its part referred to fire action or design: Eurocode 1 (Actions on Structures) – Part 1.2. Actions on Structures Exposed to Fire: This part deals with the fire loading, thermal and mechanical actions on structures exposed to fire, and thermal actions related to nominal and physically based thermal actions [2]. Eurocode 2 (Design of Concrete Structures) – Part 1.2. Structural Fire Design. This part deals with the design of concrete structures for the accidental situation of fire exposure. It identifies differences from, or supplements to, normal temperature design, and with passive methods of fire protection. It applies to concrete structures that are required to avoid collapse of the structure (load bearing function) and limiting fire spread beyond designated areas (separating function) when exposed to fire [3]. Eurocode 3 (Design of Steel Structures) – Part 1.2. Structural Fire Design. It describes the principles, requirements and rules for the structural design of steel buildings exposed to fire: Design of steel structures for accidental exposure to fire; Passive methods of fire protection; Applies to steel structures that are required to fulfill this load bearing function, avoiding premature collapse due to fire; Gives principles and application rules for designing structures for specified requirements in respect of the load bearing function and the levels of performance [4]. Eurocode 4 (Design of Composite Steel and Concrete Structures) – Part 1.2. Structural Fire Design. It describes the principles, requirements and rules for the structural design of buildings exposed to fire, including the following aspects: Safety requirements; Design procedures; Design aids. This part only identifies differences from, or supplements to, normal temperature design [5]. Eurocode 5 (Design of Timber Structures) – Part 1.2. Structural Fire Design. It refers to the design of buildings and civil engineering works in timber (solid timber, sawn, planed or in pole form, glued laminated timber or wood-based structural products e.g. LVL) or wood-based panels jointed together with adhesives or mechanical fasteners [6]. Eurocode 6 (Design of Masonry Structures) – Part 1.2. Structural Fire Design. Deals with the design of masonry structures for the accidental situation of fire exposure and is intended to be used in conjunction with other relevant Eurocodes. This part only identifies

differences from, or supplements to, normal temperature design [7]. Eurocode 9 (Design of Aluminium Structures) – Part 1.2. Structural Fire Design. It describes the principles, requirements and rules for the structural design of buildings exposed to fire, including safety requirements. This standard applies to aluminium structures required to fulfill load bearing functions and only deals with passive methods of fire protection [8].
The design is based on a limit state format. The safety factors are used for loadings and for materials. The values of the factors for materials are established to account for the inherent variability of the strength of the materials, and therefore to obtain an equivalent level of safety [9].
The reduced partial factors of safety are used to take into account the reduced probability of fire loading and the fact that during fire the structure will be more damaged than during the „normal conditions".

3. Structural Fire Resistance and Fire Tests

The fire resistance is a scalar representation of the ability of the structural elements to resist under fire condition and does not reflect the exact period of time after which the element will collapse. It is found by performing tests or by calculations and it shows the minimum period of time in which the element does not lose stability.
The standard fire resistance tests are more significant than the real fire and they are performed using a time-temperature curve. (figure 2) This curve is defined by a formula:

$$T = 20 + 345\log_{10}(8t + 1) \tag{1}$$

where
T – furnace temperature (^oC)
t – elapsed time (minutes)
Beams and columns are tested in furnaces [10].

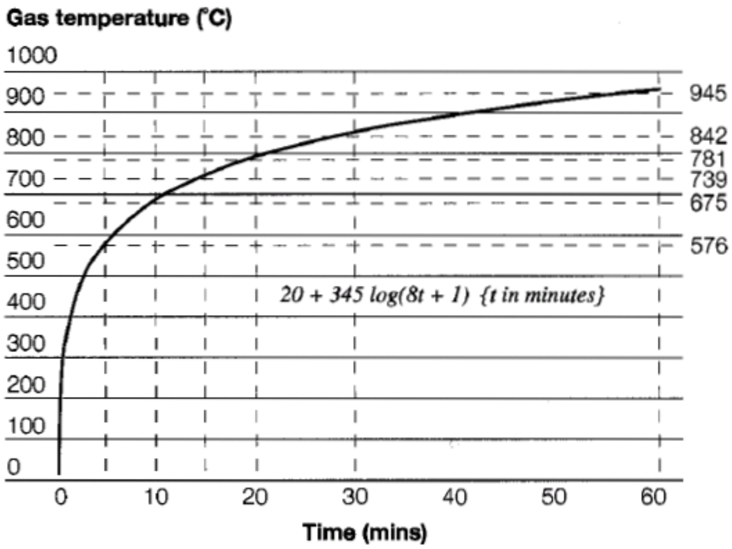


Fig. 2: Temperature-time curve (t – is time in mins) [10]

In the study of Beyler [28] all the recommendations of a standard fire resistance test "to support Performance-Based Structural Fire Engineering" are given: instrumentation, furnace construction and operation, calibration test results, specimen description, post test inspection etc. The test procedure used in this study, is ASTM E119. The paper [28] was prepared for the "Building and Fire Research Laboratory" of "The National Institute of Standards and Technology" in U.S.A.

In the following figure which shows the temperature – time curve for a natural fire we can observe that the real fire is not so severe as the gas fueled furnaces tests. (See figure 3) "Flashover" is defined as the moment in which the full compartment contents are engaged in fire.

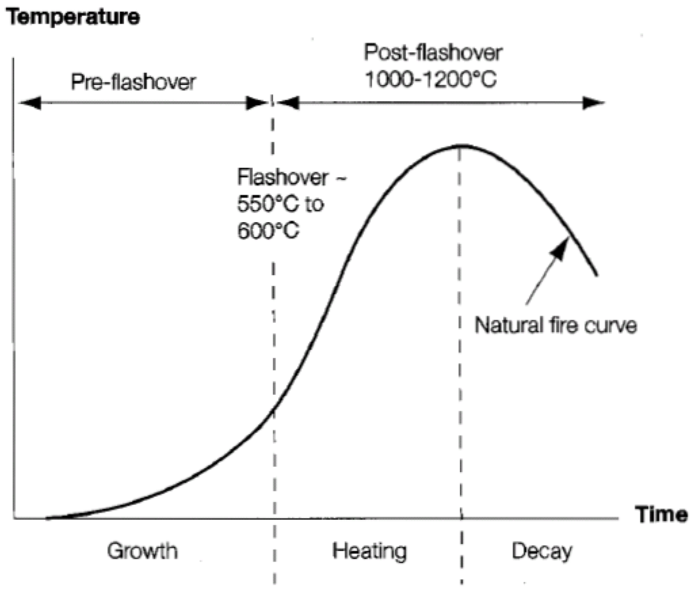


Fig. 3: Temperature-time curve in the natural fire [10]

The fire resistance may be satisfied by the use of tables and/or rules.

In the British Steel Programme major fire tests in multi-storey buildings (see figure 4) were made indicating for instance that composite floor beams have a "significant fire resistance which may lead to the elimination of passive fire protection requirements for such members" [17].

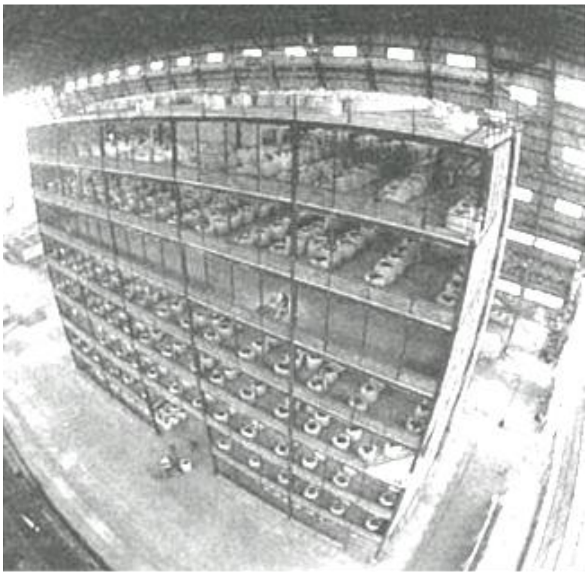


Fig. 4: Steel framed building

This building is identical with a real building and "with the proposed fire tests the intention is not to introduce idealized conditions but to consider realistic scenarios both in terms of loading and of compartment design" [17].

Several tests were presented: [17]

- heating a 9 m spanning composite beam with no fire protection by means of a gas-fired furnace. The result was indicating an excellent fire resistance.
- the structure supporting the fourth floor of the multi-storey building (two internal and two edge columns, three composite beams). The result: the two internal columns suffered severe local distortions in the exposed connection areas; the external columns suffered no significant deformations; the beams have a good fire resistance.
- another test was performed on a corner compartment of the structure having dimensions of 10m x 7.5m. (See figure 5) The maximum floor deflection was of 365mm.

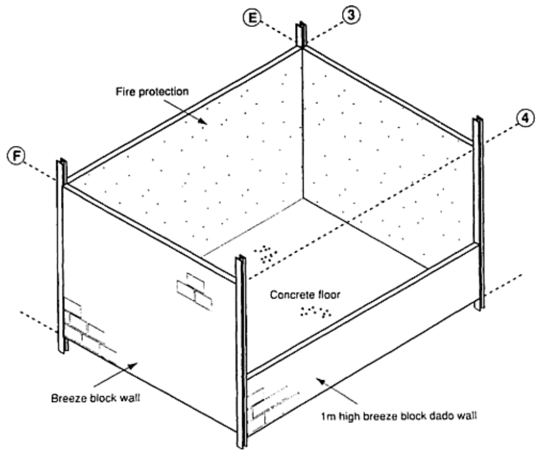


Fig. 5: Corner fire test layout

- at the Cardington LBTF was built a compartment measuring 8.6m x 5.5m x 3.9m high and inside this compartment a number of 21 fire tests were realized. One of them was conducted on a loaded steel frame. The dimensions of columns are 3.5m and the beam has a span of 4.5m. They wanted to verify the consideration that “the performance of a frame in fire would be better than its individual members” [17]. The scheme of the test is presented in figure 6:

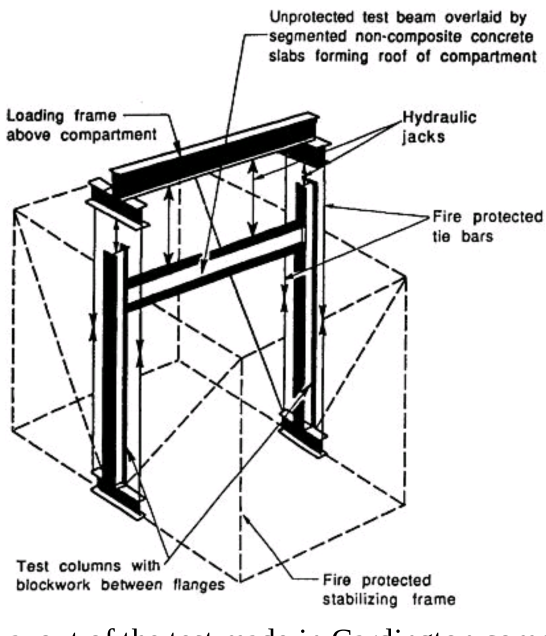


Figure 6: Layout of the test made in Cardington compartment.

The columns have concrete blocks between their flanges (a method to improve the fire resistance, see above) and the beam was left unprotected. The beam is loaded gravitationally. The individual fire resistances of members are known by previous tests. It was, in fact, proven the above consideration [17].

4. Fire Design according to Eurocodes

4.1. Actions

Thermal and mechanical actions are obtained from [2]. In addition to Eurocode 1 we choose the value of 0.7 for the emissivity of the concrete surface; [3] the emissivity of the steel surface 0.7 and for stainless steel 0.4 respectively. [4] "For surfaces of wood, wood-based materials and gypsum plasterboard the emissivity coefficient should be taken equal to 0,8". [6] For clean and not covered surfaces of aluminium the emissivity coefficient is 0.3 and for dirty or covered surfaces (for example by soot) it should be taken 0.7, [10].
The thermal actions are described by the net heat flux to the members which is the energy per unit time [W/m^2] that is absorbed by a heated surface. This parameter may be obtained from the operation of standard furnaces, and used in more general analytical models [10].
On the surfaces exposed to fire the net heat flux is determined by summing the fluxes by radiation and convection:

$$h_{net} = h_{net,r} + h_{net,c} \tag{2}$$

The component produced by convection is computed according to the following formula [2]:

$$h_{net,c} = \alpha_c (\theta_g - \theta_m) \tag{3}$$

where
α_c – heat transfer coefficient by convection;
θ_g – gas temperature near the exposed member;
θ_m – surface temperature of the member.
The component from radiation is deduced from [2]:

$$h_{net,r} = \phi \cdot \varepsilon_m \cdot \varepsilon_f \cdot \sigma[(\theta_r + 273)^4 - (\theta_m + 273)^4] \tag{4}$$

where
ϕ – the shape factor;
ε_m – the emissivity of the member surface;
ε_f – the emissivity of fire;
σ – Boltzmann Stephan constant (= 5.67 $\cdot$ 10^{-8} W/m^{2}K^4);
θ_r – fire radiation effective temperature (^{o}C);
θ_m - surface temperature of the member. (^{o}C)

In order to obtain significant effects of the actions $E_{fi,d,t}$ during fire, the mechanical actions are combined as for "accidental" situation computation. The representative value of the variable action Q_1 may be considered the value of the quasi-permanent action $\psi_{2,1}$ Q_1 [2].
In the case we do not need to take into account in an explicit way the indirect actions of fire, the effects may be determined by a simplified rule, analyzing the structure for combined actions as above only for t = 0. To simplify even more the above prescriptions, the effects of actions may be derived from those computed for normal temperature:

$$E_{fi,d,t} = E_{fi,d} = \eta_{fi} \cdot E_d \tag{5}$$

where
E_d – computation value of significant effects of actions derived from fundamental combination according to EN 1990;
$E_{fi,d}$ – constant computation value during fire;
η_{fi} – safety factor. [2]

When data presented in table are specified according to a reference loading level, the level of loading corresponds to:

$$E_{fi,d,t} = \eta_{fi,t} \cdot R_d \tag{6}$$

where
R_d – computation strength of the element under normal temperature;
$\eta_{fi,t}$ – load level for fire design [2].

4.2. Design Values of the Material Properties

In all Eurocodes except the one that treats the design of timber structures (Eurocode 5), the thermal properties for thermal analysis are specified in two cases: when the increase of the property is "favorable for safety" (case A) and when the increase of the property is "unfavorable for safety" (case B):
We have:
case A

$$X_{d,fi} = X_k(\theta) / \gamma_{M,fi} \tag{7}$$

and case B

$$X_{d,fi} = X_k(\theta) \cdot \gamma_{M,fi} \tag{8}$$

and the strength and deformation properties for structural analysis [3][4]:

$$X_{d,fi} = k(\theta)\, X_k / \gamma_{M,fi} \tag{9}$$

where
$X_k(\theta)$ is the characteristic value of a material property in fire design, generally dependent on the material temperature;
X_k is the characteristic value of a strength or deformation property (e.g. f_{ck} and f_{yk}) for normal temperature design;
$k(\theta)$ is the reduction factor for a strength or deformation property dependent on the material temperature;
$\gamma_{M,fi}$ is the partial safety factor for material property in fire design [3] [4].

The thermal and mechanical properties of concrete and steel reinforcement [3] and steel [4] the partial safety factor for fire design should be taken as: $\gamma_{M,fi} = 1.0$

In the case of timber fire design the design strength and stiffness parameters shall be determined from [6]:

$$f_{d,fi} = k_{mod,fi}\, f_{20} / \gamma_{M,fi} \tag{10}$$

$$E_{d,fi} = k_{mod,fi}\, E_{20} / \gamma_{M,fi} \tag{11}$$

where:
$f_{d,fi}$ – the design strength in fire;

$E_{d,fi}$ – the design modulus of elasticity or shear modulus in fire;
f_{20} – the 20 % fractile of strength at normal temperature;
E_{20} – the 20 % fractile of modulus of elasticity at normal temperature;
$k_{mod,fi}$ – is the modification factor for fire – it takes into account the reduction of strength and stiffness at high temperature;
$\gamma_{M,fi}$ – the partial safety factor for timber in fire [6].

4.3. Structural Fire Design

The structural fire design may be carried out by the following three methods according to the *Eurocode 2 – Design of Concrete Structures* and *Eurocode 4 – Design of Composite Steel and Concrete Structures*:

- detailing according to recognized design solutions (tabulated data);
- simplified design methods for specific types of members;
- advanced design methods for simulating the behavior of structural members, sub-assemblies or the entire structure.

According to *Eurocode 3 – Design of Steel Structures* and *Eurocode 9 – Design of Aluminium Structures* the following three methods may be applied:

- simplified design methods which give conservative results;
- advanced design methods in which engineering principles are applied in a realistic manner;
- methods based on test results.

According to *Eurocode 5 – Design of Timber Structures* the methods to apply are:

- simplified rules;
- advanced calculation methods.

In *Eurocode 6 – Design of Masonry Structures* the procedures to use in order to find the fire resistance of masonry walls are:

- by testing;
- by tabulated data;
- by calculation.

The method of using tabulated data is very easy to use but the domain of applicability is very restrained due to the geometrical conditions imposed to sections. This method is used only for reinforced concrete and composite steel and concrete structures [12]. The tables are developed on an empirical basis confirmed by tests. More specific tabulated data may be found in the product standards.
The simplified rules are based on simplified formulas (for steel) or abacs (for reinforced concrete).
The advanced calculation method is performed with the help of advanced calculation models calibrated with test results. (see chapter 1 of this study)

4.4. Cases not specifically covered by Eurocodes

Various design cases may be encountered that are not specifically treated by Eurocodes 3 and 4 part 1.2., like:

- provisions to take on the bracing system;
- beams with shelf angles;
- portal frames;
- water – filled structures;
- fire resisting walls;
- roofs and ceilings;
- re-use of steel after a fire.

5. Conclusions

The probability of a fire extreme enough to produce the collapse of the building is, according to [2], considered low. In Eurocode 1 the actions are presented and their grouping, too. The dead loads have a partial factor equal to 1.0 and the live loads have a partial factor reduced down to 0.5. They are combined as an "accidental" grouping of actions. Thermal actions are expressed in terms of the net heat flux.
The strength or deformation properties of the materials are increased/reduced in order to obtain design values more "favorable for safety".
Calculation methods are either simplified calculation methods based on conservative hypothesis or more advanced calculation methods in which engineering principles are applied in a realistic manner to specific applications. In what concerns the software that uses advanced calculation models, this must be confirmed throughout pertinent tests for each category of element being designed.

6. References

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CHOICE
Participation at the international architectural competition for the urban development project of Vabaduse Square/ town Rakvere/ESTONIA

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Abstract

The City of Rakvere/Estonia has organized an open international architectural competition for the design of Arvo Part's Hall by reconstruction of Rakvere's St Paul's church (nowadays sport venue) and planning the surrounding area called Vabaduse Square (Freedom Square).
Our team participated with a project with the entry name CHOICE. We focused on making a new urban identity for Vabaduse Square, which should, at the same time, emphasize the main qualities of the site, of the town, in the existing circumstances. We proposed a continuous space for music and cultural events, variable, fluctuating, to give a chance to everybody to make its CHOICE in a large scale of convenient occasions (for an event or another, at a chosen time, regardless the number of participants) in both urban and architectural approaches.
We conceived the extension of St.Paul's Church to assure a total flexibility for the desired activities and open possibilities for the improvised ones. The entire building can be used as a whole, or can be divided in independent parts, each one with different time schedules, different entrances, and different users. We have chosen an austere expression for the inside and outside appearance of the building, in agreement with the austere Lutheran image of the church. Like in a church, the main importance is focused on light and verticality, as an optimistic aspiration for a bright future. A reversible relation both in the urban and in the architectural space, like "mirror in mirror" (the title of one of Arvo Part compositions), summarizes our approach.

Rezumat

Primaria oraşului Rakvere/Estonia a organizat o competitie internationala de arhitectura pentru proiectarea Salii de Concerte Arvo Part prin extinderea spatiului bisericii Sf. Paul din Rakvere(astazi sala de sport), şi pentru amenajarea urbana a Pietii Vabaduse (Piaţa Libertăţii).
Choice (alegere) este titlul pe care colectivul nostru l-a ales pentru denumirea proiectului cu care am participat la acest concurs. Am realizat o noua identitate urbana pentru piata Vabaduse, bazandu-ne pe evidentierea si punerea in valoare a principalelor trasaturi specifice locului, orasului, circumstantelor existente. Propunere noastra a constat in crearea unui spatiu continuu pentru muzica si evenimente culturale, variabil, fluctuant, pentru a permite tuturor utilizatorilor posibilitatea de a opta (CHOICE) pentru abordarea dorita a spatiilor (indiferent de dimensiunea grupului de participanti, de alegerea facuta pentru un eveniment sau altul, de momentul ales).
Pentru extinderea Bisericii Sf. Paul am conceput un spatiu continuu, flexibil ce asigura posibilitati multiple de diversificare atat a activitatilor necesare, cerute prin tema concursului, cat si unor alte evenimente , improvizatii ad-hoc. Intreaga constructie functioneaza atat ca un spatiu total, cat si

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ca spatii diferentiate, cu programe diferite, intrari diferite, utilizatori diferiti si concomitenti. Am optat pentru o expresie arhitecturala minimalista atat in interiorul cladirii cat si in exteriorul ei, in concordanta cu austera imagine a a bisericii luterane.Asemanator spatiului bisericii, principala atentie ne-a fost indreptata spre accentuarea luminii si a verticalitatii, sugerand astfel o atitudine optimista cu speranta intr-un viitor stralucit. Demersul nostru a urmarit crearea unei relatii reversibile intre spatiul urban si spatiul arhitectural, ca „oglinda in oglinda" (titlul unei melodii compuse de Arvo Par)..

Keywords: urban design, cultural events, new identity, choice, continuous space, convenient occasion, static, dynamic, porch , hierarchy, rhythm, minimalist language.

1. Introduction - description of the competition theme.

This paper is the illustration of the project proposed by the team from the Faculty of Architecture and Urban Planning, T.U. in Cluj-Napoca, Romania, in the international architectural competition for the design of Arvo Part's Hall by reconstruction of Rakvere's St Paul's church and planning the surrounding area called Vabaduse Square (Freedom Square).
The requirements of the competition theme where one side to restore the noble nature of the Vabaduse Square (planned at the beginning of the 20-th century as Rakvere's main square) due to the public buildings which are situated here: the kindergarten - currently a Steiner school, the German secondary school - currently a dental polyclinic , the Estonian secondary school and the St.Paul's Church - currently a sport hall. On the other side, the St.Paul's Church , the work of the important Estonian architect Alar Kotli ,erected in 1940, must be restored and extended with the Arvo Part's Hall, a concert hall for 400 people. The church/concert complex with the annex and the reorganization of Vabaduse Squar would make Rakvere an"excellent place for organizing concerts and would facilitate the enlivening"/2/ of the cultural and educational life of the town.

2. Basic concept.

MOTTO: SPACE is like MUSIC.
MUSIC and SPACE are endless.
MUSIC and SPACE means movement.
You can hear and see different aspects of the existence depending the distances put between and the direction of the approach.
The controlled movement is like the gamut for music, it creates rules for behaviors and impressions.
Perspectives, frames, surprises (sequences) can be appropriated as a matter of CHOICE you make in your movement/1/.

THE CONCEPT:
To make a new identity to an urban space/building means to develop the existing circumstances, to emphasize their main qualities.
Main qualities of Rakvere Vabaduse square:

- static, classical composition, homogeneous architecture of public buildings by scale and rhythm
- „Rakvere's nobility centre ... open to the sun and air"/2/;
- the „optimistic atmosphere that focuses on a bright and clean future"/2/;
- the slope of the soil;
- end perspective to convergent streets and park alleys.

Main qualities of St. Paul's Church:

- symbol of Estonia's freedom;

- important work of the Estonian architect Alar Kotli/2/ (outstanding importance in the architectural history of Rakvere as well as Estonia);
- a distinctive character in the surrounding urban space (urban landmark).

Main quality of the cultural life in Rakvere:
- the Arvo Part Festival, -the most significant cultural event of the town in the international context.

THE ATTITUDE:
We propose a continuous space for music and cultural events, variable, fluctuating, to give a chance to everybody to make its CHOICE of the approach desired, in the desired team, in a desired purpose, at the time desired, in a large scale of convenient occasions.
We propose a continuous space from outside to inside, from top to bottom, from dark to light, from static to dynamic, from monumental to friendly, from hidden to exposed, from historicity to contemporaneousness.
We use in that purpose a lapidary, minimalist language, inspired by the calm, classical urban and architectural surroundings and by the mystic minimalism that characterize the works of Arvo Part.
A reversible relation both in the urban and in the architectural space, like, "mirror in mirror", summarizes our approach.

3. The public space .

The public space proposed means CONTROLLED MOVEMENT.

Because the huge scale of the existing Vabaduse square, we divided it in different spaces, appropriated to convenient occasions.

Figure 1 – Proiect - aerial view of the Vadabuse Square

At the top of the slope we organized the CULTURAL SQUARE (Fig. 1), a plain surface, facilitating open-air events and performances, in front of and expanding the spaces of the new multi-functional public building (The Arvo Part Hall).
An artesian water fountain, surrounded by a half recessed area to allow sitting places, mark the intersection of the axis of: Vabaduse Street and Tuleviku Street, articulating also a "water way" to the "future public building".

This cultural square, underlined by public buildings on three sides, is the official side of Vabaduse Square, contains also different sculptures (in front of the Polyclinic to animate this closed building). At important events, this Cultural Square can occupy all the space bordered by the Policlinic, the Secondary School and Arvo Part Hall.

Emphasizing the slope, THE STEPS: moving up and down people sitting, laying down, singing, jumping, skating. This friendly area is underlined / protected by alignment of trees next to Voimla and Vabaduse streets. A descending view to the “future public building”, situated along the Koidula Street, brings closer both spaces, engaging common activities. The “water way” finishes in a rectangular fountain, a calm surface to reflect the building in front of it, or to refresh the tired legs of pedestrians in hot summers. (Fig. 2).

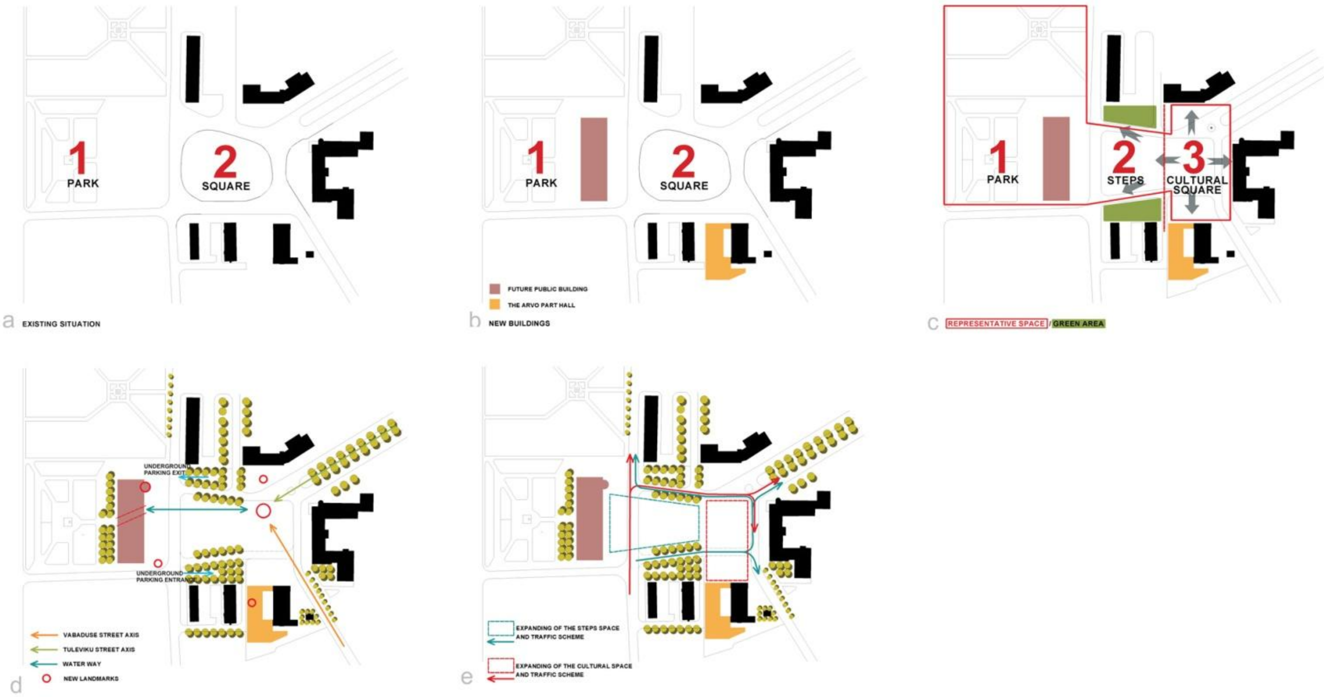

Figure 2 –Park, steps, square – adaptable sequences for cultural events in Vabaduse Square - concept schemes.

TWO GREEN AREAS give chances to privacy for dwelling houses. They remove the inconvenient relation between garages and other back spaces with Vabaduse square. In the same time, these green areas hidden entries to an underground two level parking serving the entire square. We increased the distance between Voimla Street and the dwelling houses, in order to develop the green area in this part of Vabaduse Square, modifying the line of this street.

We amplified the impact of the existing buildings bordering Vabaduse Square, reducing the distance of their perception by introducing smaller landmarks – fountains, sculptures, trees, in order to graduate and enrich the perspectives. We interrupted the huge open perspective from the secondary school to the park, placing the “future public building “ along Koidula street. The passages proposed through this building to the park emphasize the impact and the scale of the World War II Memorial. The long perspective to War of Independence Monument is divided by the green area and the alignment of trees creating different frames for its perception. (Fig. 3)

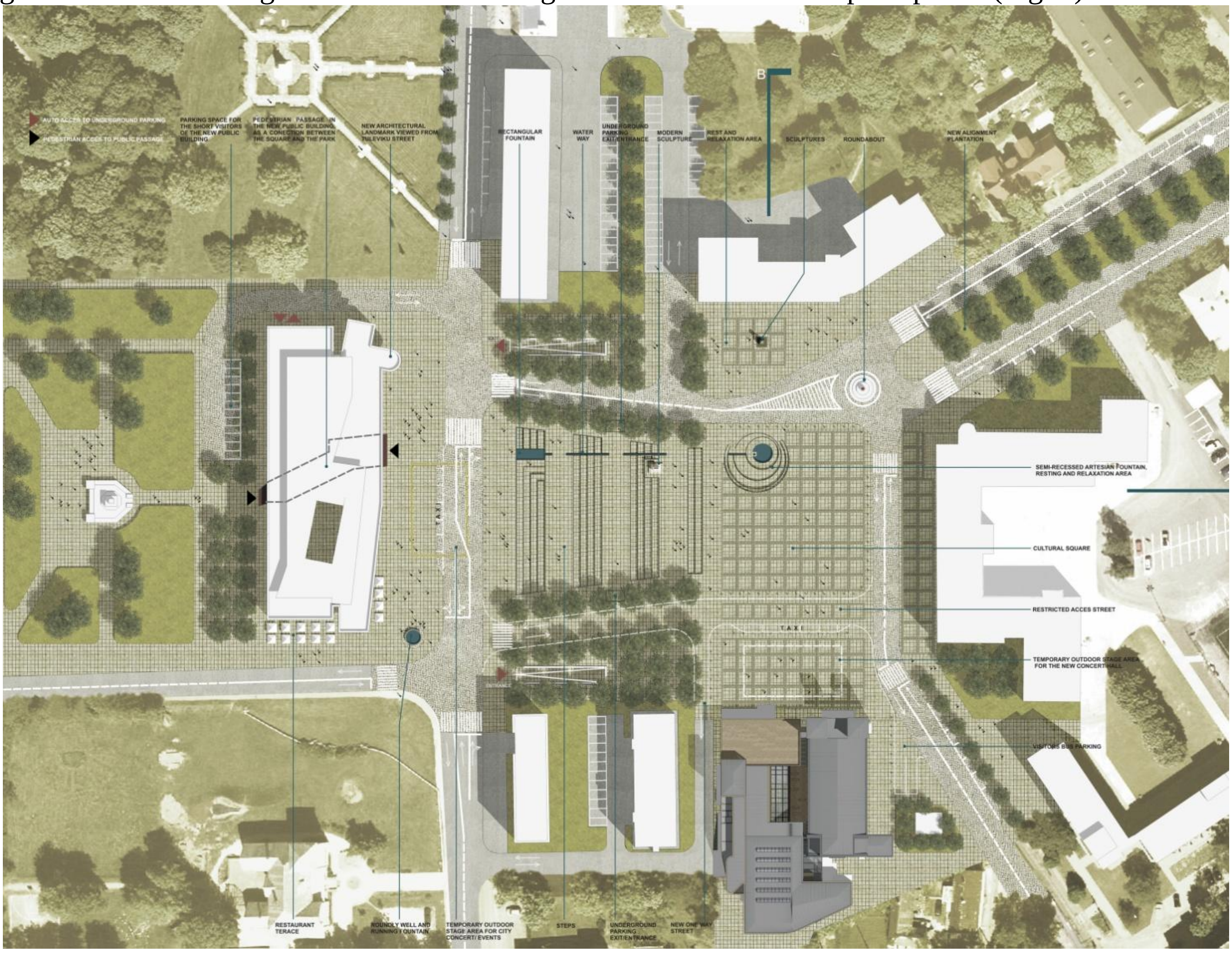

Figure 3 – Vabaduse Square Site Plan

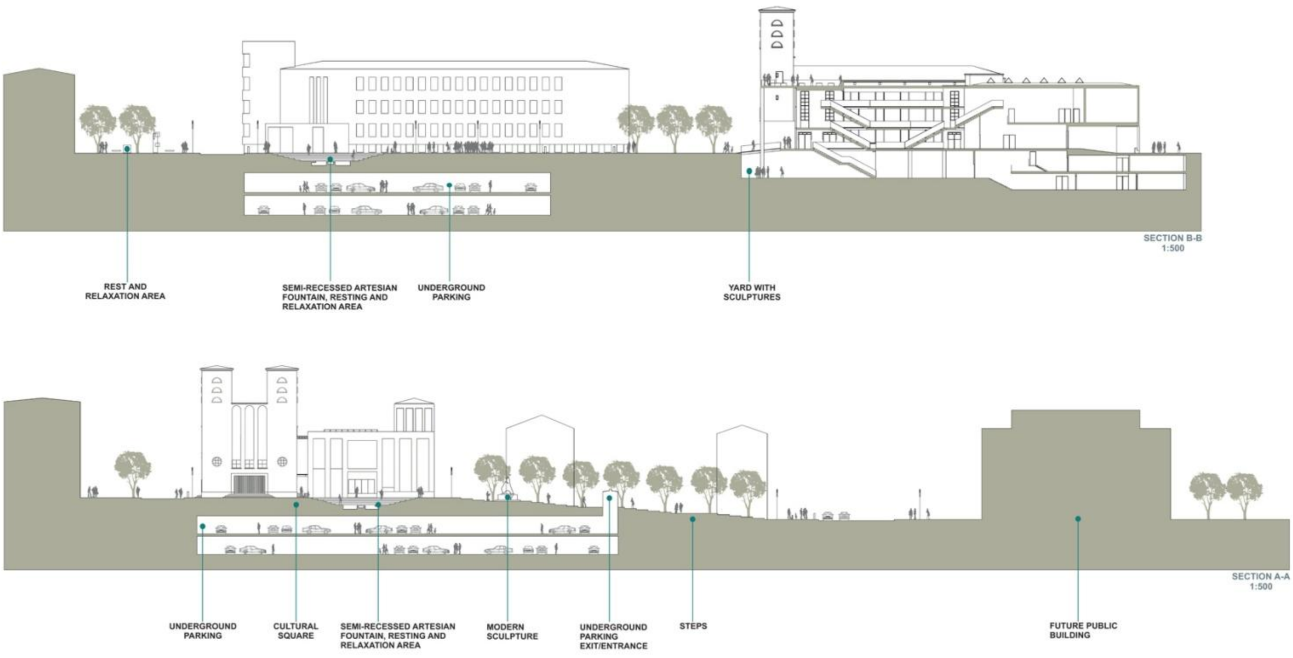

Figure 4 - Cross Sections - Vabaduse Square and the new Arvo Part Hall.

The Arvo Part Hall, extending St.Paul's Church, continues the alignment of this building and complete the Voimla Street front side.
Its austere porch open the entire space of the multifunctional public building to the cultural square, proposing a democratic connection between inside and outside. An open "yard with sculptures "underlines the porch and creates a descendent perspective to underground art happenings. A pedestrian path connects the secondary entrance of the building to Kindergarten Kaur. (Fig. 4)

In concordance with the "optimistic atmosphere" of Vabaduse Square, we proposed the use of solar colors for pavements; stones in different ochre nuances cover its surface and create a harmonious relation with the trees. Bronze sculptures erect here and there to animate the spaces. We tried to balance a reasonable ratio between mineral and vegetal surfaces, in agreement with all seasons.

4. Arvo Part Hall architectural design

We conceived the extension of St.Paul's Church using, like in music, a leitmotif:

THE PORCH, a succession of pillars.

Figure 5 – Main entrance in Arvo Part Hall – the porch /view from the square.

We chose it because of its transparency and rhythm. A porch separates spaces and at the same time unifies them, makes hierarchies but establishes friendly connections. It is an open but protective space. Its scale gives significance to a space, from representative and imposing to cozy and intimate. It can have different depths; it can inspire rhythm for openings (windows and doors).and it is also a strong landmark/3/.

We conceived the extension of St.Paul's Church like a CONTINOUS SPACE.

We chose making the CONTINOUS SPACE to assure a total flexibility for the desired activities and open possibilities for the improvised ones. The entire building can be used as a whole, or can be divided in independent parts, each one with different time schedules, different entrances, and

different users.
The ground floor (Fig. 6) is designed to surprise and to inform about everything can happen in this building. A symphony of spaces is revealed from the main entrance (from Vabaduse square). The impressive space, the opened hall, is announced from outside by the great porch and by the half hidden yard with sculptures. Light, space and richness of images gladdens the eye and the important surprise is to rediscover inside the St.Paul's Church. The image of the church accompanies the visitor at each floor, offering different perspectives. At this floor, a continuous space connects the small concert hall (introduced in the church in a mobile system that allows the alternation of

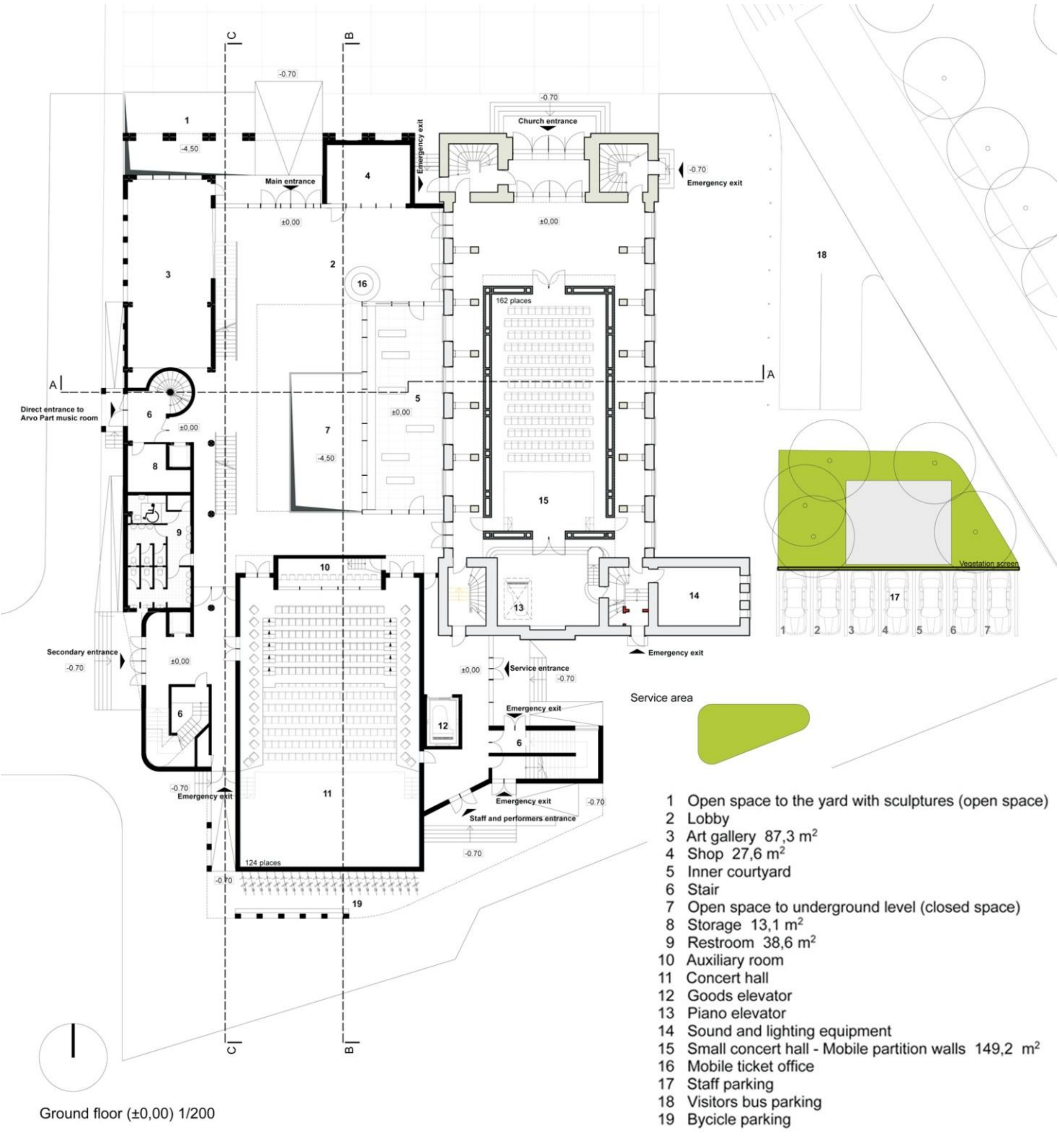

Figure 6 - Ground floor

performances with the celebration of the religious service) with the balcony of the big concert hall, the art gallery and the music shop. A continuous movement is possible in case of big events, but at the same time each space can be closed and has its separate entrance. Lateral, near the art gallery we organized an entrance for small musical events performed at the third floor, in the Arvo Part music room (like a rise in a church tower, a spiral stair case and an elevator assures independency for different time schedules).

In the basement, a foyer for the big concert hall offers opportunities for art events and continued with the yard with sculptures can stirs up the attention of people passing the square.
The big concert hall is located here, in order to assure a perfect acoustic of high level, in accordance with the contemporary exigencies, and preferring not to occult the space of the church. At the same time, this level offers a wonderful ascendant perspective to the Church.
We organized hear also, on two levels, the necessary technical spaces, at the heights of -3.50 (Fig.7) and – 4.50 (Fig. 8).

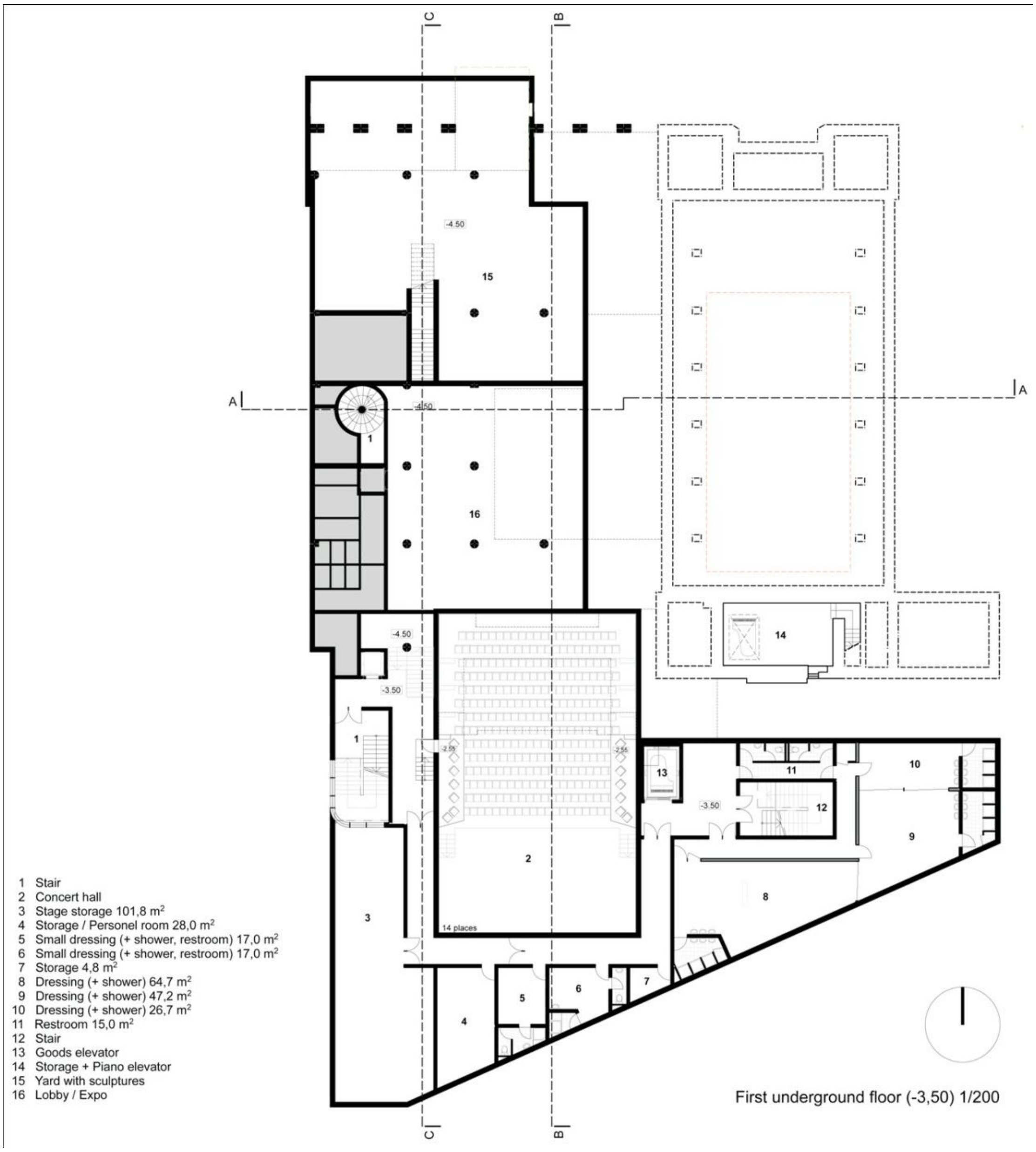

Figure 7 - First Underground floor (-3.50)

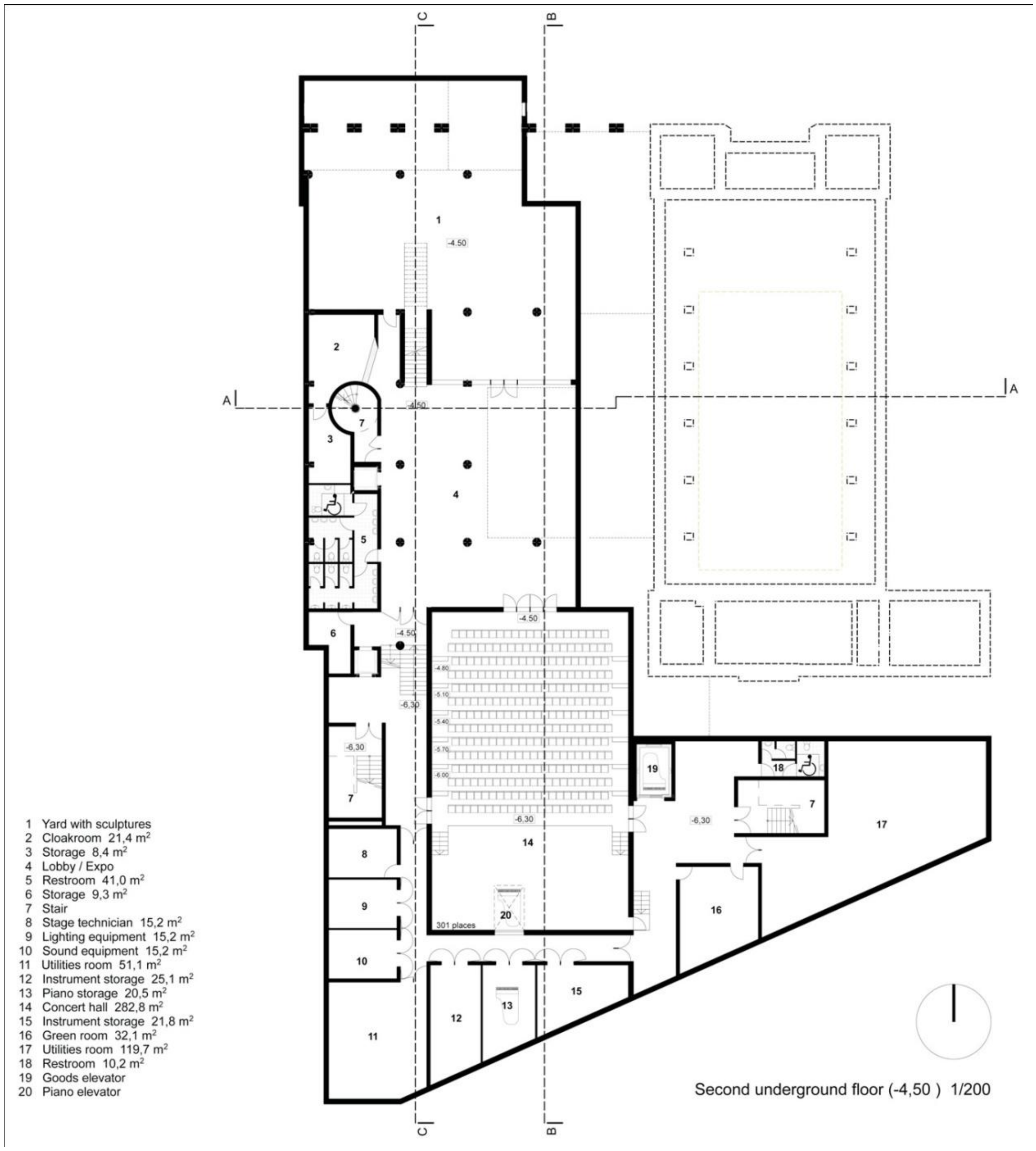

Figure 8 - Second Underground Floor (-4.50)

The upper levels : the first(Fig. 9) and the second (Fig.10), organize the practice rooms for dance and choir, the caféteria room, spaces for exhibitions in connection with the balcony of the church and the loggia that opens this part of the building to the Vabaduse square.
Fluent platforms, stairs, elevators assure the continuity of the space, accessibility, flexibility for different uses, offering permanently original images of the church. These spaces can be also accessed from the lateral side directly by a separate entrance assuring opportunities for different time schedule programs.

At the top, the third floor (Fig.11) exposes the "Small Temple" as an important landmark of the Vabaduse square and of the city, lodging the Arvo Part music room, with its intimate and exclusive access. An open terrace and an open galleria expand the small music room to the square ant to the city; from here we can see the roof and the towers of St.Paul's Church and a large panorama to the square, the park and the city. Long mobile benches allow also here open air performances.

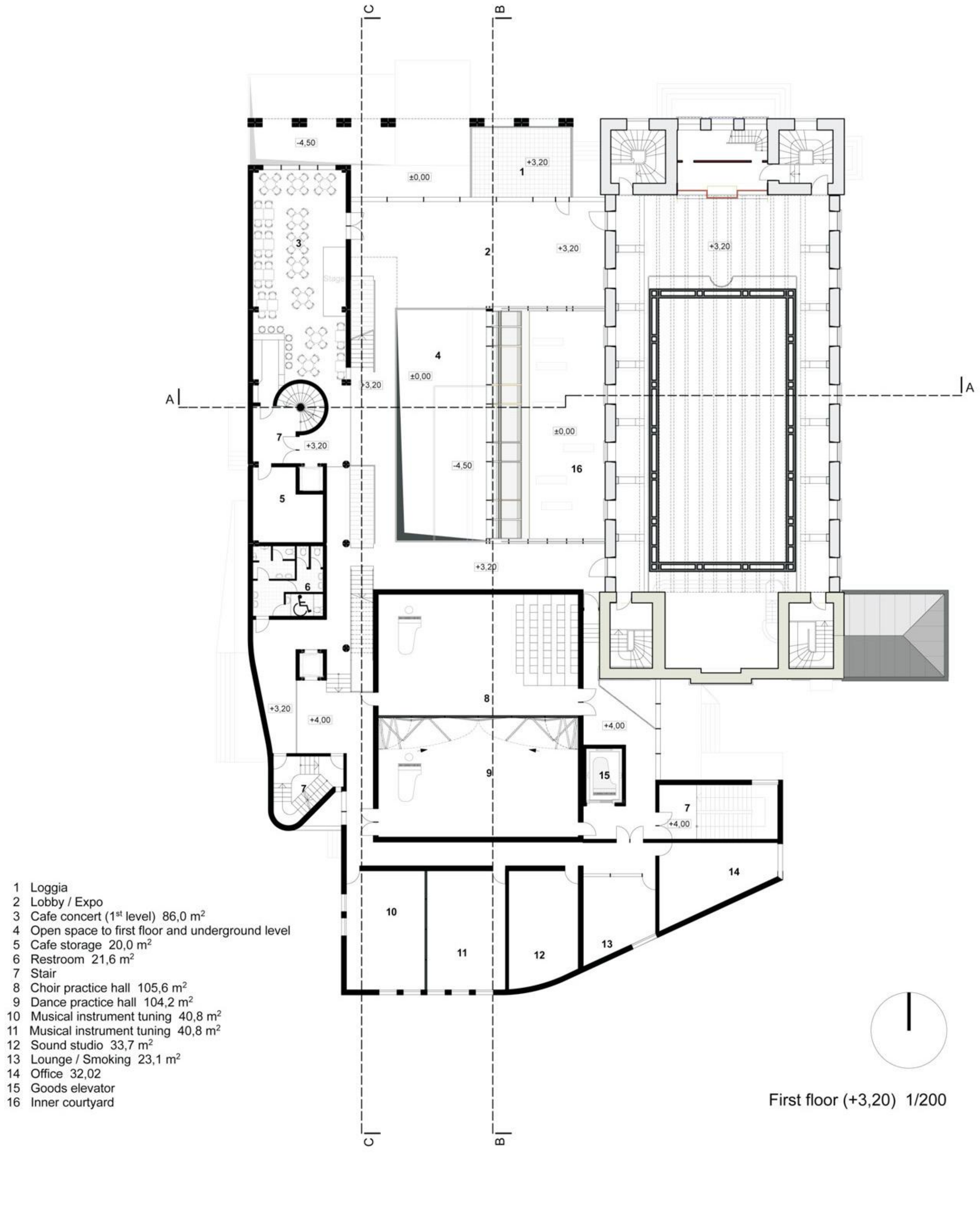

Figure 9 - First Floor

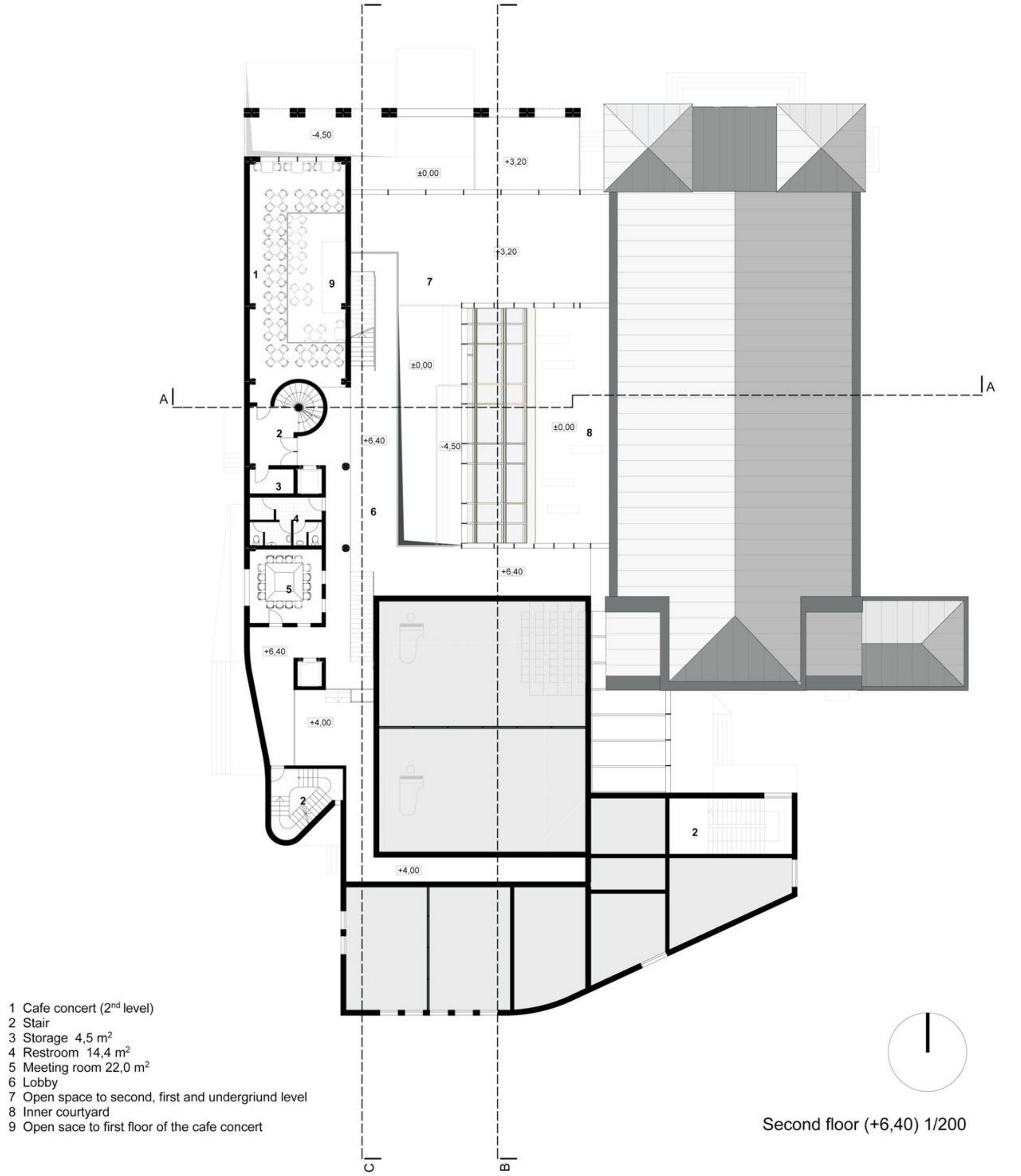

Figure 10 - Second Floor

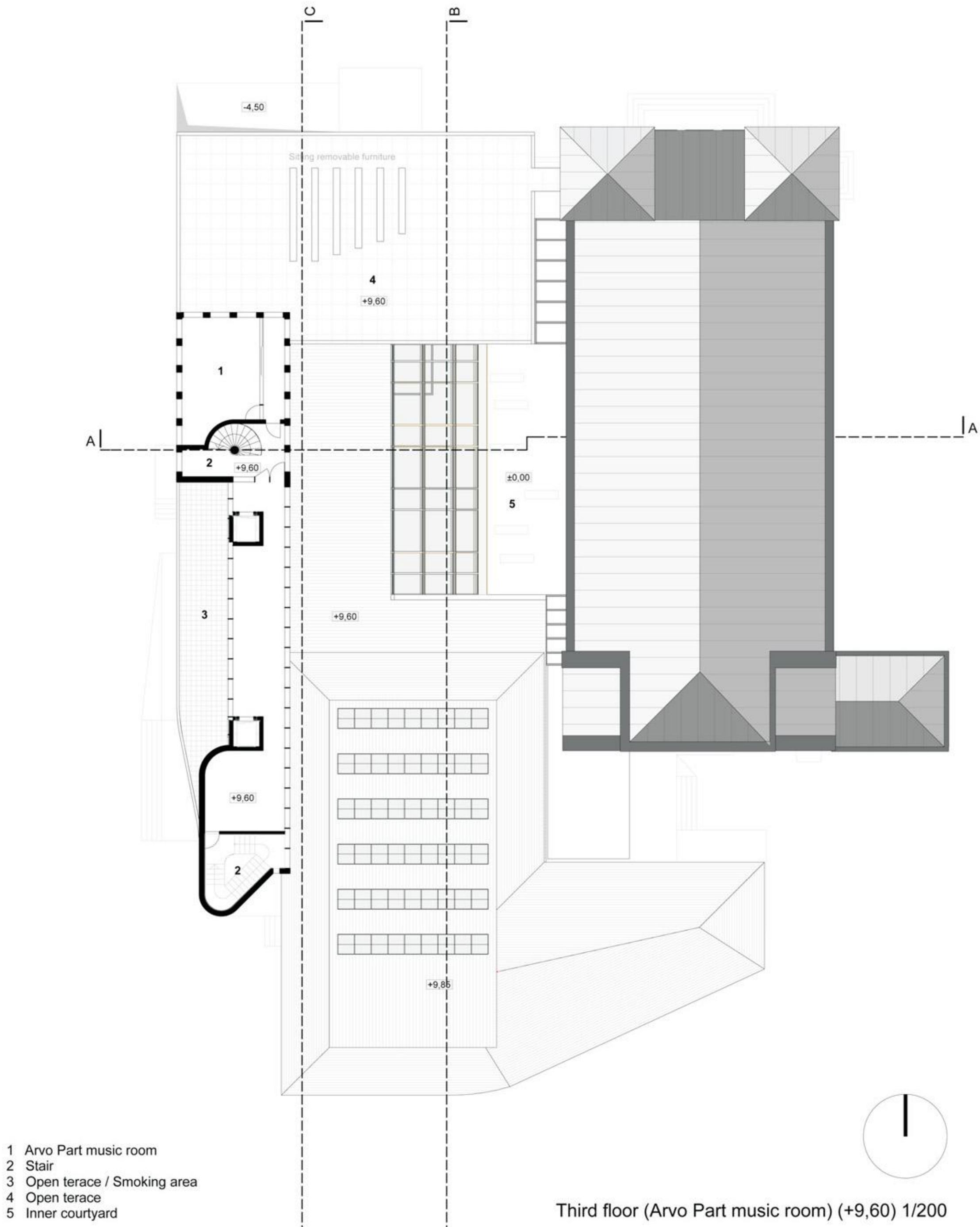

Figure 11 - Third Floor

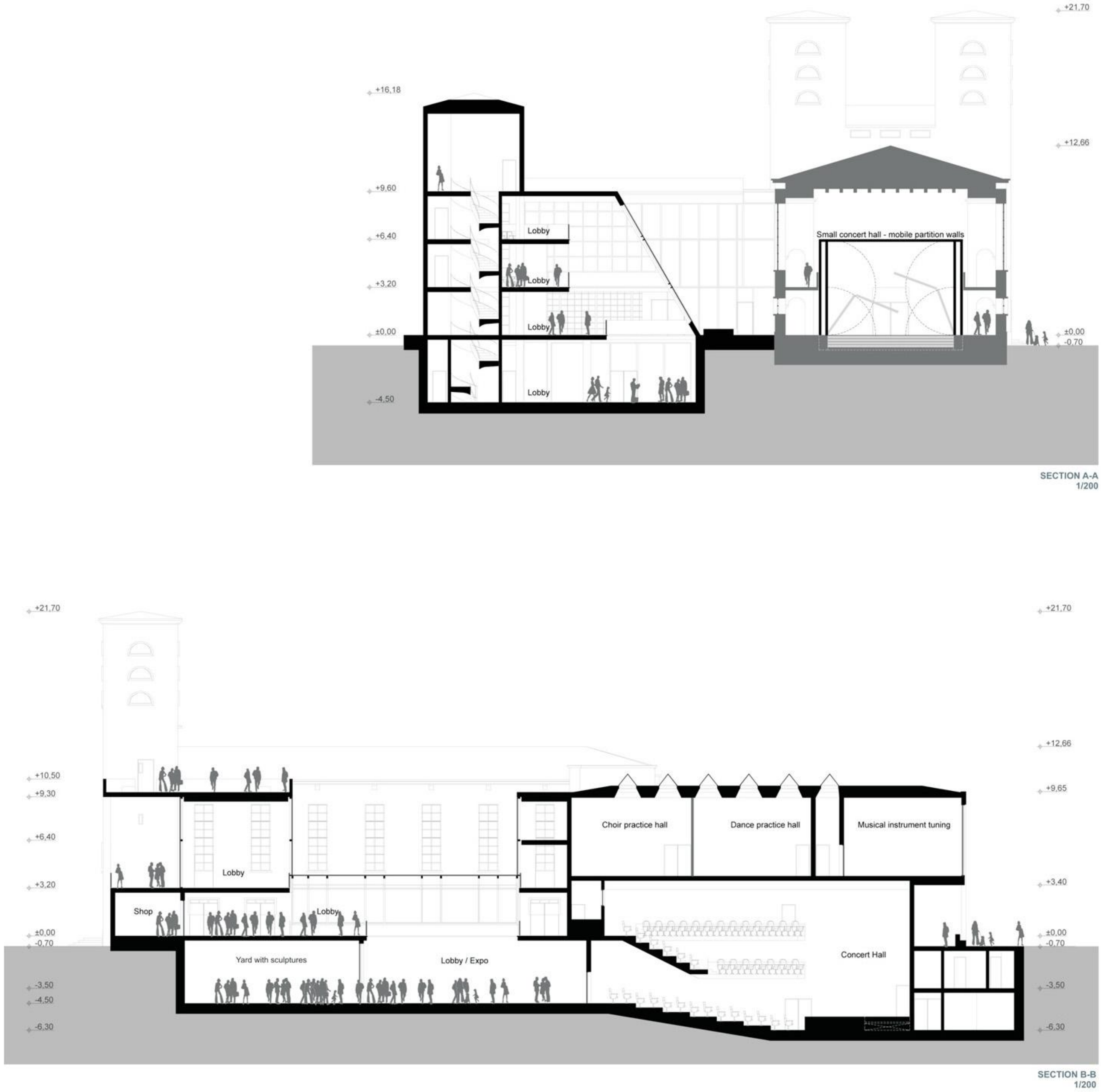

Figure 12 –Main cross sections /up- transversal section with the church St.Paul, down- longitudinal section.

Opposite to the main entrance, “in the back”, all levels contain the rights annexes for the services deserving all main functions: offices, restrooms, rooms for musicians and musical instruments, near the service area, as we can see in the description of the spaces contained by each level and near the secondary entrance.

We chose an austere expression for the inside and outside appearance of the building, in agreement with the austere Lutheran image of the church. Like in a church, the main importance is focused on light and verticality, as an optimistic aspiration for a bright future.
Noble materials, marble for pavements, plastered white walls and big glass openings (windows and doors) assure a serene image for the internal spaces. We proposed the same treatment of the surfaces in the restored church as in the new building, in order to make a pacific path for users, to assure the coherence of the interior of the whole building. All the concert halls are conceived as “wood worlds” (protection for acoustic reasons).

The big porch of the main entrance and all the exterior walls and pillars are covered with bricks of the same color as of the church walls.

A lapidary but reach in details world, an atmosphere of calm and peace dominates the external image (Fig. 13 - Facades).

Figure 13 - Facades

Figure 14 - Interior and exterior views of Arvo Part Hall.

We chose this minimalist style/4/ as a very contemporary tendency of today/5/, specific for our times and in agreement with the surrounding area and the sentiment that inspire the mystic minimalism of Arvo Part music. (Fig. 14)

5. Short description of technical aspects: the construction system.

The construction system is a simple one: reinforced concrete for pillars, beams and floors, brick covered concrete walls and/or bricks walls, wood used as acoustic protection.
We proposed also wood frames for windows, and wood made doors, like in the restored church.
We preferred the utilization of construction materials with good maintenance possibilities (wood, stone, concrete, glass). All materials proposed are also of high thermal efficiency.

6. Conclusion

Each international architectural competition is a good opportunity for the interaction of different attitudes and approaches of thinking and making architecture. Depending on experiences and on special needs specific for each lieu and representing the desires of each collectivity, the proposed themes are always a challenge for participants. Put in an inedited situation, the participant should anticipate not only the right answer for solving real needs but also to think and understand in a totally different cultural background, the way of developing prospective attitudes. For such reason, these competitions are very good opportunities also for real researches in the field of architecture and urban design.
Our project proposes an open, flexible urban and architectural solution for hosting very diverse cultural events, a contemporary space which rehabilitates the unfinished Vabaduse Square, transforming it in an open cultural site, revalorize and reestablish the initial function of the beautiful St.Paul’s Church.

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Financing a research activity through national programs

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Abstract

The scientific research activity represents a major factor in the socio-economic development and an engine of the economic and social progress; science and technology are key components of modern life and they help countries achieve their economic and social goals, achieve sustainable development.
In scientific research we distinguish between fundamental research, applied research and research for development and innovation. Thus, if knowledge refers to rules and principles, we can consider it a result of basic research, expressed in publications. If knowledge refers to procedures or to the application of this knowledge in specific contexts, as a result of the basic research, then we can discuss about the applied scientific research, whose result appears in publications.
Key words: financing, research, development, national programs, innovation.

Rezumat

Activitatea de cercetare ştiinţifică reprezintă un factor important care contribuie la dezvoltarea economico-socială şi un motor al progresului economico-social; ştiinţa şi tehnologia sunt componente de bază ale vieţii moderne şi ajută direct statele în realizarea obiectivelor economice şi sociale, în realizarea dezvoltării durabile.
În cercetarea ştiinţifică se face distincţia între cercetarea fundamentală, cercetarea aplicată şi cercetarea pentru dezvoltare şi inovare. Astfel, dacă cunoştinţele se referă la reguli şi principii, putem vorbi despre cunoştinţe rezultat al cercetării fundamentale, exprimate în publicaţii. Dacă cunoştinţele se referă la proceduri sau la aplicarea cunoştinţelor rezultat al cercetării fundamentale la contexte specifice, atunci putem discuta despre cercetarea ştiinţifică aplicată, al cărei rezultat apare în publicaţii.

1. Introduction

The purpose of the research activity is to create knowledge, this knowledge being expressed, primarily, in publications. If the knowledge is in accordance with the legislation in order to appear in scientific publications which can be sustained by invention licences and prototypes, invention licensing and prototypes registration assure a commercial protection of the published knowledge. So, research - development produce knowledge expressed in publications,

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sustained by invention licensing and prototypes registration, assuring also a commercial protection.

Innovation is the activity which leads to the creation, the assimilation and the development of research-development results in the socio-economic area. The transformation of knowledge in publications, of licensing and prototypes (research-development) in economic and social assimilated technologies and services (innovation), is not the main purpose of the scientific research, but it is related to the socio-economic field, and by that we mean transfer and knowledge distribution. Besides the classic instruments tied to the education, such as universities, specialized instruments were built to ease knowledge transfer from research-development to the socio-economic field. Among these instruments we can find: (1) spin-offs; (2) technological platforms; (3) specialized organisations, where researchers and businessmen work together. But, in this last case, the researcher's activity is often accounted for an educational, economic activity, instead of a research activity. Therefore, the involvement of the researchers in the innovation activity doesn't measure out as a scientific activity but as a socio-economic one, which gives them a profitable material gain.(Văcărel I *et al*., 2006)

The importance of the research activity occurs in a lot of official documents: thus, the scientific research activity represents the highest degree of human resources development, through both the constant learning/training they demand and the achieved results. But yet, although indispensable for development and for the resolution of society's global problems, the scientific research was the first research generated in Romania, being considered a luxury in comparison with the crisis of the moment.

2. National financing programs for the research-development activity

The National Authority for Scientific Research (NASC), as a governmental organ, responsible for the elaboration and the harmonisation of national policies for research – development and innovation next with the European policies, wants to assure the connection between the objectives of the scientific community and of the business world from Romania, and the typical priorities concerning science and technology in the European Union, as well as the dynamics of their evolution.

The support for planning and implementing the RDI politics promoted by NASR in the after-adhesion period was given by:

* The National Strategy RDI for the period 2007-2013 (National Strategy), ratified by GD no. 217/2007;

* The National Plan for Research, Development and Innovation for 2007-2013 (National Plan), ratified by GD no. 475/2007.

The National Strategy's main goal until 2013 is the recovery of the existent differences towards the European countries level, and the preparation of the RDI system in order to identify and consolidate, through international paths, partnership and competition, those areas in which Romania can excel.

The strategic objectives established by the National Strategy in 2007-2013 are:

1. The creation of knowledge, respectively obtaining high scientific and technological results, internationally competitive.
2. Increasing the competitiveness of Romanian economy, by innovation with impact on the economical operators' level and knowledge transfer in the economic practice.

3. Increasing the quality of social life, by the development of solutions, including technological solutions which can generate direct benefits to the society.

The most important financial programs for research, completing the National Plan, are the nucleus Programs, the Sectorial Plans and the Fundamental Research Programs of the Romanian Academy.

2.1. The National Plan for Research, Development and Innovation 2007-2013

The National Plan is the main instrument for the implementation of the National Strategy. This was ratified by Government Ordinance nr. 475/2007, legislative act which decides the rules and the implementing principles, the constituent programs, the investment model and the budget – 15 billion lei for 2007-2013, the monitoring procedure, as well as the evaluation and impact indicators.

The National Plan for Research, Development and Innovation for 2007-2013, called further on – the National Plan II – NP II, represents the main instrument used by the National Authority for Scientific Research (NASR) to implement the National Strategy for RDI.

In the creation of NP II, we were referred to the role of the national research development – innovation system, which is to develop science and technology in order to increase the economic competitiveness, the improvement of the social value and the growth of knowledge by turning to account and increasing the action environment.

NP II focuses on achieving all three strategic objectives of the national system RDI:

1. Creating knowledge, obtaining high scientific and technological results, globally competitive, in order to increase the international visibility of Romanian research and the following transfer of these results in the socio-economic practice.
2. Increasing competitiveness of the Romanian economy by innovation, with a high impact on the level of the economic agents and the knowledge transfer in the economic practice.
3. Increasing the social value, finding technical and scientific solutions which sustain social development and improve the human condition. The achievement of these three general objectives will be the result of a long term vision regarding the national system of RDI and its importance in the society.

In this way, the RDI system could become the engine of the development of the world of knowledge in Romania, being capable to sustain performance through innovation in every domain which contributes to the welfare of the citizens and also to achieve scientific excellence, internationally recognized.

To respond to this challenge, the RDI system will be defined by an approach, first of all to Romanian companies, then to the international scientific field, the society's needs and the educational system.

General Principles in the foundation of NP II

In the creation of NP II we took into consideration the results received following the current state analysis of the national economy and of the research, development and innovation system, and the strategic objectives originated by the National Strategic Reference Framework for 2007-2013, by the international conditions, as well as by the perspective of Romania's integration in the European Union. Also, we took into account the experience accumulated in the organisation and the progress of the RDI activity as a result of the course of the National Program of RDI in 1999-2006, of the Grant Program for scientific research in 1995-2006, of the

Excellence Research Program (CEE X) in 2005-2006, as well as the results obtained by widespread consulting of the national and international scientific community, realised in 2005-2006, as part of the first widespread national foresight exercise in science and technology.

NP II is based on the following principles:

I. The transformation of public expenses of RDI in investments for RDI, meaning:

1. Correlating the objectives regarding the strategy-programs-instruments, including the level of the monitoring indicators 2007-2013, is pursued.
2. Funding and refunding the resources of the program are realised as part of an investment model which demands an evaluation ex-before, monitoring, evaluation ex-after and rectifications during the progress of NP II. The application of the investment model falls within the competence of the NASR, and the specific methodologies are ratified by Decree of the President of NASR.
3. In the evaluation of the management institutions which implement programs and projects, we will supervise professionalism and transparency as key elements for success.
4. Simplifying the funding procedures and activity discount within the framework of the projects is pursued.
5. The evaluation of the programs NP II is done every two years as part of the evaluation of the implementation. The evaluation of the NP II implementation is realised preferably internationally, the evaluation report being made public.
6. The Grant Allocation by NP II is based on competition. The specific criteria for the projects evaluation is included in the information brochures of the programs and it's ratified by Decree of the President of NASR.

II. The focus on the public investment in RDI, which presumes:

7. Allocating public resources for RDI projects has the following destinations:
   - Domains which touch the boundary of knowledge, in which Romania has resources and results or which should support the priorities of the applied research;
   - RDI's priorities, obtained following the national foresight exercise;
   - supporting the innovation.

8. Sustaining the progress of the RDI system will be guaranteed by the investment in the human resources development, infrastructure of the research and the increase of the international widespread degree.

III. The Training of the Private Sector

9. The economic sector will be permanently checked in order to determine its needs for development
10. The public investment in RDI will stimulate the interest in RDI's activity in the economic field, its partnerships with public entities, with consequences upon the growth of RDI's private investment.
11. The development of the infrastructure and services for technological transfer for a better use of RDI's results in the economic field, with the protection of intellectual property.

IV. Wide access and proper evaluation

12. The access to the funding via National Plan II will be made with discrimination, by competition.
13. International researchers will have access to participate in national projects.

14. Innovative companies will have wide access for RDI project funding, especially the ones in collaboration with university and institutional research.
15. The procedures regarding competitions will be simplified.
16. The projects will be monitored continuously, and the final reports will be made public. The failures exceeding the risks accepted by the research contracts falls within the competence of the project team.

V. The regional aspects of NP II are additional to the instruments which use the structural funds for RDI.

17. The approach of the regional dimension of RDI objectives takes into account the complementarity with structural funds, focalising on the identification of RDI's demands on a regional level, the stimulation of human resources development and capabilities of research on a regional level and the promotion of the collaboration on a regional level.

The Program *Resurse Umane (Human Resources)* has as a main objective increasing the number of the researchers and their professional performances, and as secondary objectives:

I. Increasing the number of PhD-students and post-PhD-researchers
II. Increasing the appeal of the research carrier, especially for eminent graduates
III. Attracting Romanian researchers which have high performances abroad
IV. Creating centres of excellence around scientific reputed personalities and internationally recognized
V. Increasing national and international mobilities for researchers
VI. Stimulating the creation of centres of excellence
VII. Improving RDI unities' management.

This program has the following action channels:

1) The creation and perfection of researchers by master's degree and post-master's degree
2) Funding the integration projects of researchers from abroad in the Romanian system
3) Sustaining the excellence for researchers with scientific performances and excellence schools internationally recognized
4) Funding national and international mobilities for researchers
5) Training in research and innovation management
6) Awarding excellent results in research

The Program *Capacitati (Capacities)* has as a main objective the development of research capacities and the CDI opening system to the international scientific field and the national socio-economic field, and as secondary objectives:

I. Increasing the degree of the use of research infrastructure
II. Developing the research infrastructure
III. Developing the infrastructure of the information and scientific documentation
IV. Turning to account the potential and RDI's resources in a regional plan
V. Sustaining the science-society dialog
VI. The participation of RDI entities to national and international scientific organisations
VII. The participation of RDI entities to international research programs

The acting channels of this program are the followings:

1) The creation and support of national interest infrastructures for research;
2) Fund assurance in order to stop and preserve some complex national interest installations;

3) The consolidation of the research infrastructure with multiple users;
4) The consolidation of the offering capacity and the use of "experimental services;
5) The improvement of the quality of scientific magazines, especially by sustaining the co-editing with important international publishing companies;
6) The support of scientific and exponential manifestations;
7) The improvement and extension of the infrastructure and of communication services for research;
8) The development and the acquisition of data base specific to the RDI system;
9) The development of on-line resource access for documentation;
10) The promotion of communication and the consolidation of the role of science in society;
11) The elaboration of forecasting studies in science and society
12) The preparation and stimulation of the participation to international programs for research;
13) The support of the participation of RDI entities to organisations and international research programs;
14) Introducing thematic common calls in partnership with other countries;
15) Sustaining the representation of Romania in organisations and international research programs;
16) Assuring consultancy and assistance activities for the state research authority;

The Program *Idei (Ideas)* has as main objective obtaining high scientific and technological results, similar to those on an international level, reflected by the development of the visibility and international recognition of Romanian research, and as secondary objectives:

I. The continuous improvement of visible performances on an international level in the domains in which Romania has a potential for research and in which there were results comparable to other EU countries;

II. The development of the domains in which Romania has an interest to develop research scientific activities with real contributions to the growth of the quality of knowledge, the technical and technological development and the improvement of life quality.

The acting channels of this program are the following:

1) Sustaining fundamental, frontier and exploratory scientific researches;
2) The organisation of "exploratory workshops" designed to identify the knowledge recesses unexplored;
3) Making calls for international collaborations for fundamental, frontier and exploratory research projects.

Partnerships in primary domains program's objective is the growth of RD's competitiveness by stimulating partnerships in primary domains, specific to technologies, products and innovative services in order to solve some complex problems and to create implementing mechanisms, and the secondary objectives are the followings:

I. The growth of the capacity of RDI's sector for Technology of Information and Communication in order to sustain the society and the economy based on knowledge;

II. Increasing the technological competence and promoting the knowledge and technologies transfer in the energy field, according to conditions of quality, safety in supply, and following the principle of sustainable development;

III. Creating clean products, processes and technologies and taking into account the waste management system;

IV. The scientific substantiation and technology development in order to preserve, rebuilt and consolidate the ecological and biological diversity;

V. Knowledge development in the field of land use planning in a sustainable manner;

VI. The optimisation of methods of disease prevention, medical therapies development and the efficiency of the public health system;

VII. Promoting the sustainable agriculture, increasing alimentation security and safety of products;

VIII. Developing biotechnologies with an impact on the quality of life and the economic development;

IX. Developing new materials, products and high value added processes;

X. Increasing the competitiveness of Romania in the research and space technologies domain;

XI. Identifying and solving the primary social problems concerning education, living and workplace in order to obtain local, regional and national development;

XII. Increasing the competitiveness and creativity, the development of organisational culture in the economic systems, public administration, education and research, in the sanitary system and the military one;

XIII. Taking into account and developing the national culture patrimony;

XIV. Decreasing socio-human discrepancies/discriminations and regional disparities.

The acting channels of the program are the followings:

1) Supporting RDI projects on thematic directions;
2) Supporting RDI projects on primary themes established on the basis of consultation;
3) Supporting the research networks.

Innovation Program has as a main objective to increase the ability of innovation, technological development, and assimilation of the results of the research into production, in order to improve the national economy's competitiveness and the growth of the quality of life, and the secondary objectives are the following:

I. Consolidating the ability for innovation of companies and consolidating their contribution to the creation of new products and markets based on the knowledge results value;

II. Stimulating partnerships between economic agents and research entities;

III. Developing technological transfer abilities in universities;

IV. Stimulating the capacity of absorption of RDI results by SMBs;

V. Implementing strategic agendas elaborated on the basis of technological platform;

VI. Creating and developing innovation infrastructures;

VII. Developing the infrastructure and quality management.

The acting channels are:

1) Creating products and technologies at the initiative of economic agents;
2) Creating and/or developing the innovation infrastructure: scientific and/or technological parks, centres of technological transfer, brokerage centres as well as knowledge stores, technological incubators;
3) Sustaining the offer of innovation services support;
4) Supporting the development of the infrastructure for quality attestation;
5) Supporting the accreditation of laboratories for essays and analysis;
6) Supporting the implementing and the development of the quality management system;

7) Supporting the creating and the development of innovative networks;
8) Supporting the activity for technological platforms;
9) Organising awarding contests by fields of activity with innovative potential on a national scale.

Sustaining the institutional performance Program has as main objective supporting this performance by assuring the continuity and the stability of RDI entities' activities, with a view to implement personal development strategies, elaborated in agreement with the RDI National Strategy, and the main objectives are the following:

I. Sustaining the institutional development in order to achieve excellence;
II. Sustaining the international competitiveness of the Romanian RDI system.

The specific actions of this program are the guarantee of a multiannual finance, by competition, for activities without any economic degree/nature, which can allow the RDI unity to run the development program on a medium term and achieving a level of performance which contributes to the attraction of additional financing sources.

2.2 Nucleus

The nucleus Programs are settled by the GD no. 137/2003 regarding the scientific research and the contracting, financing, monitoring, and evaluating modalities of these programs are settled by the methodological Norm no. 6/2003.

This norm determines the fact that these programs are approved by the state authority for research – development, especially the National Authority for Scientific Research (art. 11). In these norms it's also stipulated a 30% payment in advance of the payments that need to be done within the framework of the program (art. 16), but lately it's often stipulated that this advance payment should be by 90%.

Also, this norm is settling that the value of the grant from the budget cannot surpass 50% of the revenues of the research-development activity, but some changes are made by legislation and, at the moment, the value of the grant from the budget can achieve 75% of the revenues of the research-development activity.

Every year, the values for nucleus programs are foreseen in the NASR budget, and these values can increase or decrease as part of budgetary rectifications which are made in one financial year.

2.3. Sectoral Programs

These sectoral programs are settled also by OG nr. 57/2002 and the applying methodological Norms are referred to HG nr. 1266/2004 which provides:

"Art. 1

(1) Sectoral plans are elaborated by the public financing authorities, so called contracting authorities, with the agreement of the Ministry of Education and Research, in quality of state authority for research-development.

....

(3) Sectoral plans contain programs and projects for research-development of high interest for the domain in question, so called projects, which engages on the achievement of the objectives from the development strategy of the field coordinated by the contracting authority, and which are foreseen to be realised in a determined period of time, according to the public funds allocated in this purpose.

(4) for the final touch of the sectorial plans structure, the contracting authorities can demand project proposals from structures representatives of the coordinated field."

This way, in 2004 appeared the Ordinance nr. 4728 of the Ministry of Education and Research which approves the sectorial Plan for research-development of the ministry in question, with the following general objectives: the development of the infrastructure of research-development and of the ministry's ability to elaborate, implement and evaluate politics, strategies and programs on a national level in research, development and innovation.

The specific objectives are:

* improving the performance of the activity of research-development unities' activity;
* developing the structure TIC for research;
* developing the ministry's ability to ground, elaborate, update and to communicate politics, strategies and programs;
* developing the instruments used by the ministry for planning, following and evaluating the realisations of politics, strategies and programs on a national level in research, development and innovation.

Among the estimated results we mention:

- the increase of the number of projects realised in partnership;
- the assurance of some internal sources of high scientific competence and technological expertise, in the reference fields
- the elaboration, implementation and evaluation of politics, strategies and programs on a national level in the field of research-development and innovation (RDI);
- integrated information and operational system which will allow the efficiency of the main parts of the research projects progress: competition, evaluation, progress, finalisation, and technological transfer, as well as evaluating the research's results in the zone of its final beneficiaries;
- the dissemination of the results with high degree of application and multiplication and of those of strategic interest;
- information in real time of the results obtained in the fields of research, development and innovation;
- the identification of the legislative instruments (laws, regulations, norms, codes) specific to RDI system;
- the identification of the specific indirect mechanisms (tax and income taxes system) of support in the RD domain and of encouragement of the RD results application;
- the growth of performances of national networks specialised for research-development.

In 2008, the Ordinance no. 668 of the Ministry of Work, Family and Equality of Chances appeared, and it also approves a sectoral plan for research-development for 2009-2012.

This sectoral plan is structured in 5 afferent programs to the main domains of responsibility of MMFES:

1. The program "Work Market"
2. The program "Pensions"
3. The program "Social Inclusion"
4. The program "Workers Mobility"
5. The program "Security and health in work"

The specific objectives of these programs were: increasing general administrative ability of the MMFES and the institutions in its suborder, under its authority or coordination, to develop and apply strategies and integrated action plans designed to increase the level of the occupancy of the work force on a national, regional and local level; the consolidation of the administrative ability of the public pension system; promoting some adequate politics for social inclusion in order to increase the degree of social cohesion; creating an integrated management system of the workers' migration fluxes and perfecting the social protection systems for immigrant workers; the continuous improvement of the security and health level in work has as a purpose to develop security and health measurements at work in order to improve work conditions, as well as the assurance of cars and individual safe protection equipments for workers.

The estimated results of this sectoral plan were: the administrative ability consolidated for the elaboration, implementation, monitoring and evaluation of politics (strategies, programs, legislation) in the workforce, including the actualisation of the Classification of occupations in Romania on scientific basis, in function of the evolution of the workforce market; informatics statistical system developed for public pensions and prognosis regarding the finance sustainability evolution of the public scheme for pensions and alternative models of insurance of its sustainability; the consolidation of the national system of coordination and monitoring of social inclusion politics; research for estimating the dimensions, trends and migration effects on the workforce market and the development of the integrated management system of the migration of the fluxes of workers; the improvement of the legislative framework and the security and health level in work.

2.4. Romanian Academy's fundamental research programs

In the framework of the Romanian Academy's institutions there are fundamental research activities, especially as part of the master's degree or post-master's degree program. Basically, funding these research studies is made from funds from the Operational program for Human Resources Development as part of structural grants.

3. Conclusions

The economy based on knowledge is defined by the speed with which new knowledge turns into economic effects. For this to happen, the circuit research-education-innovation needs to be much more stressed on an institutional level, so that we arrive at a symbiosis between activities. In many advanced countries the interaction research-education is favourable by the fact that research is developed considerably in the university. In these conditions, big companies invest in universities (supplies, project funding), and small innovative companies gravitate around universities (scientific parks, technological centres). Even when there's a system of applied research institutes, directed to the collaboration with the industry (like Frauhofer institutes in Germany), the interaction with education is clear (professors manage the research laboratories "populated" mostly by PhD students).)Răzvan V. F. *et al*., 2006)

There's the immediate chance of collaboration between universities and institutes, using the research made by PhD students as linking points. The universities' PhD students are often employees of the institutions, but their master degree's activity doesn't have any connection with their research at their workplace. What is significant is the orientation of foreign companies in production and research-development activities in Romania towards the collaboration with universities, most of all in order to insure their human resources. It's a "timid" start, if we compare it with everything happening in the world. It would be necessary that research becomes a goal of the collaboration.(Paul Vass A., 2010)

Unfortunately, the Romanian industry was systematically demolished in the past 20 years (including by denationalisation), and small innovative companies – subscribed to public grants – don't usually succeed to produce something or to provide competitive services. As a result, with small exceptions, "the internal market" for the research-development sector offer is very poor, and "the external market" is hardly approached. There are few institutes in Romania which collaborate with foreign important companies and this happens, usually, within European projects.

In the last couple of years, the institutes acquired last generation equipments, upgraded in comparison with the national companies' equipments. However, the collaboration with the industry cannot limit itself to punctual services, but it has to take into consideration product development and advanced technologies, internationally competitive. And this is possible only by strategic orientation, sustained by public or private funds investments (for instance, the Renault investment, delayed because of the world crisis). The European approach puts into consideration INCDs who provide, basically like the branch industries, commercial services. (Curaj A., 2000)

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