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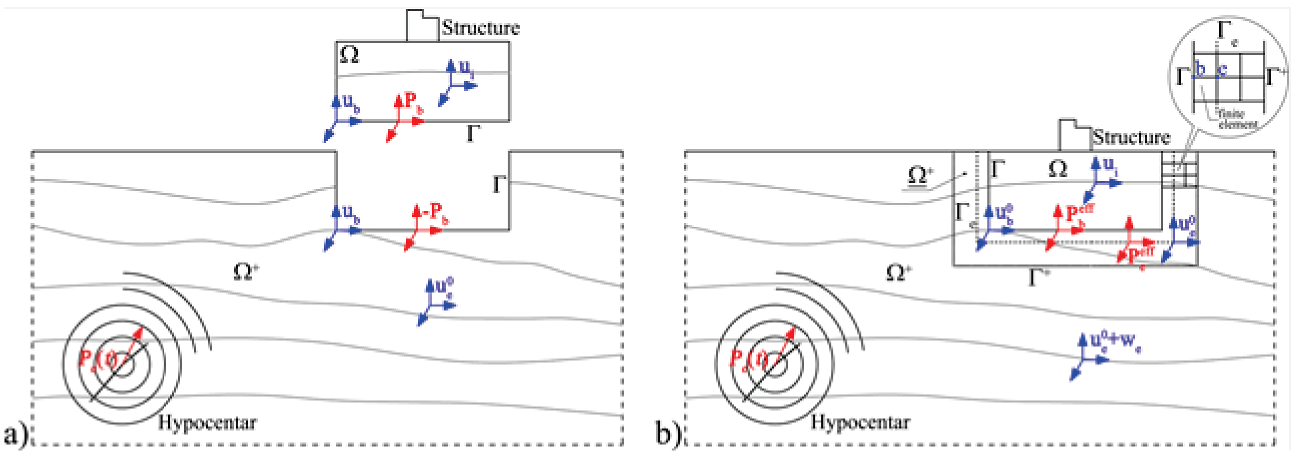

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Building Materials and Structures aims at providing an international forum for communication and dissemination of innovative research and application in the field of building materials and structures. Journal publishes papers on the characterization of building materials properties, their technologies and modeling. In the area of structural engineering Journal publishes papers dealing with new developments in application of structural mechanics principles and digital technologies for the analysis and design of structures, as well as on the application and skillful use of novel building materials and technologies.

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Review paper

Contribution of strain-hardening cementitious composites (SHCC) to shear resistance in hybrid reinforced concrete beams

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ABSTRACT

Strain Hardening Cementitious Composite (SHCC) is an innovative type of fibre-reinforced cement-based composite that has superior tensile properties. Because of this, it holds the potential to enhance the shear capacity of reinforced concrete (RC) beams, if applied properly. This paper presents the general and distinctive properties of SHCC as well as a literature review of topics related to the contribution of SHCC layers to the shear resistance of RC beams with and without shear reinforcement. Based on the analysed results, it is concluded that the main characteristics of SHCC are its microcracking behaviour, high ductility, and increased tensile strength (between 2 and 8 MPa) at large deformations. When used in structural elements, SHCC develops multiple parallel cracks compared to concentrated cracks in conventionally reinforced concrete. The biggest disadvantage of SHCC is its significant drying shrinkage. Although showing high variability, using SHCC as laminates with a thickness of 10 mm improves the shear capacity of hybrid RC beams, but debonding of interfaces in a hybrid system occurrs in some cases.

1 Introduction

Concrete is the go-to material for construction due to its affordability, versatility, and ease of use [1, 2]. However, even well-designed concrete structures require regular maintenance and assessment to reach their expected lifespan. Durability issues can lead to costly repairs [3], which is a concern in the era of sustainability and the circular economy. While low cost and versatility have been the driving factors for concrete's dominance, sustainable development will most likely drive our economy in the future [2]. Hence, the sustainability of construction materials will become increasingly prominent.

In order to address the challenges faced by traditional construction materials, it is necessary to explore novel materials that offer greater benefits. One such innovative material is the Strain-Hardening Cementitious Composite (SHCC), which is a special type of cement-based composite reinforced with fibres. SHCC exhibits superior crack control ability under tension [4], thanks to the fibres that bridge cracks at high ultimate tensile strain, giving it an edge over traditional concrete. Consequently, SHCC can lead to improved durability [5] that can potentially outlast that of NC.

Still, the high cost and environmental burden of SHCC make it impractical for use as the sole construction material [6,7]. A more viable solution would be to use a hybrid system that combines SHCC and traditional concrete. By doing so, the material can be used more efficiently, reducing the burden on the environment and keeping its costs competitive. One possible method is to place SHCC in the outer layers of a conventional reinforced concrete (RC) beam, thus utilizing its superior mechanical properties and potentially reducing the need for reinforcement. U-shape shells made of SHCC can be prefabricated in a concrete element factory and used as formwork for cast-in-situ concrete, which can further reduce costs. This hybrid system offers a more sustainable and economical option for construction projects.

Previous research has explored the impact of using hybrid systems with outer layers of SHCC. An experimental study on the flexural and cracking behaviour of reinforced SHCC layers in the tension zone of RC beams was conducted [5] . It was found that hybrid systems exhibit superior cracking behaviour and smaller crack widths compared to conventional RC beams. In the follow-up study, it was shown that the choice of interface property and fibre type can affect the controlled micro-cracking behaviour and resulting cracking pattern of beams. Further research on this topic is available in [8]. On the other hand, only limited experimental research on the shear behaviour of hybrid

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systems is present in the literature. Although it was numerically shown that when applied monolithically, reinforced SHCC can have increased shear capacity compared to the reference, conventional reinforced concrete [9], it is not clear if the benefits will be preserved in the hybrid system where SHCC and traditional concrete have to work together to bridge the critical shear crack.

This paper presents the general and distinctive properties of SHCC as well as a literature review of topics related to the contribution of SHCC layers to the shear capacity of RC beams with and without shear reinforcement. Shear failure in concrete members or structures is classified as sudden and brittle and thus should not be the dominant failure mechanism for structural members. It is anticipated that SHCC laminates can increase the shear capacity of a hybrid structure compared to a conventional RC structure. However, the fracture behaviour under shear load in hybrid systems is not yet fully understood, as fracture properties are influenced by factors such as concrete strength, curing age, volume fraction of fibres, aggregates, and the interface between conventional concrete and SHCC. Depending on the surface pre-treatment during manufacturing, a premature debonding of SHCC laminates from a traditional concrete core can happen, which can outweigh the benefits of SHCC application.

2 Composition and general properties of strain-hardening cementitious composite (SHCC)

SHCC is a cement-based material originally developed by Victor Li in the 1990s [10], under the name Engineered Cementitious Composite (ECC). A special micromechanics design of matrix, containing fibres and only fine particles provides a "microcrack bridging property" and improved crack control ability with multiple microcracks rather than a single concentrated crack. As a result, pseudo strain-hardening behaviour under tension, as illustrated in Figure 1, is obtained. This characteristic has led researchers to name it "Strain Hardening”. SHCC shows high deformability up to a tensile strain of around 5%. In comparison, the maximum tensile strain of traditional concrete is approximately 0.01%. Due to its decreased brittleness, SHCC is reported to be a promising repair material [4] and has increased bond strength and abrasion resistance [11].

Over the years, different mixtures of SHCC were developed, starting with a traditional one that includes Ordinary Portland Cement (OPC), fly ash, and silica sand with a relatively low fibre content of 2% or less by volume. The substitution of fly ash with blast furnace slag and silicate sand with limestone powder were applied shortly after. Variations in mixture composition considered both the inclusion of coarse sand and fine nanomaterial additives as well. In order to reduce curing time when SHCC is applied for repair, different chemical admixtures that accelerate hydration and hardening were added to SHCC. Mixtures with improved viscosity and bond strength aim to enable better workability and prevent early failure of the repair. More improvements addressing certain properties included: enhancing micro-cracking capacity and ductility by adding polystyrene beads to the mix, including recycled tyre rubber to decrease the modulus of elasticity of the material, including pre-soaked expanded perlite aggregate to reduce shrinkage, or adding super absorbent polymer (SAP) or bacteria to enhance the self-healing of microcracks. Most of these attempts aim at one of two goals: increasing the bond strength between the two materials or/and improving a certain property of the (repair) material (i.e., reducing shrinkage, reducing eigenstresses, increasing strength, triggering self-healing properties, etc.).

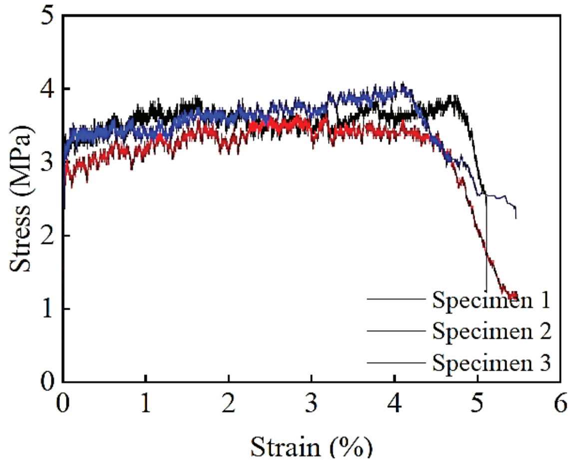


Figure 1. Example of tensile stress–strain curves of SHCC at the age of 28 days [12]

The characteristic behaviour of SHCC includes small crack widths ranging from 60 μm to 100 μm [4] and significant ductility after cracking, with a strain capacity two orders of magnitude higher than that of traditional concrete [13]. Small crack widths in SHCC offer two key advantages. Firstly, it exhibits good self-healing properties [14]. Secondly, water permeability through those cracks is similar to that of uncracked concrete [15]. As a result, it is anticipated that the service life of a SHCC will be longer than that of a NC. However, if SHCC is to be used as a complete replacement for traditional reinforced concrete, it is neither environmentally nor financially feasible for the construction industry due to the higher amount of binder and higher cost of ingredients used in SHCC. The use of Polyvinyl Alcohol (PVA), as well as other types of fibres, often increases the overall cost of SHCC; therefore, their use must be justified.

2.1 Fibres

There are many types of fibres used in SHCC. According to [13], different fibres have an influence on crack width control in hybrid structures containing SHCC and traditional concrete. The most common fibres from the literature review are Polyvinyl Alcohol (PVA) fibres, High Modulus Polyethylene (HMPE) fibres, and steel fibres (Figure 2). Attempts are also made to use natural fibres in SHCC [16].

After the formation of a crack, fibres play a crucial role in the micromechanical design by increasing the post-cracking strength. There is a common misconception that fibres prevent the formation of cracks, but in reality, the crack appears at the same load level as in traditional concrete. However, the fibres control the crack width. The small crack width in SHCC results in a longer service life of the structure since narrow cracks can slow down the ingress of chloride ions and thus better prevent reinforcement from corroding. Namely, due to the high ductility of SHCC, corrosion-induced expansion leads to controlled splitting cracks, and the probability of spalling is reduced. Cracks are very small and are filled with corrosion products, which is not the case with reference concrete [20]. Further ingress of deleterious materials is also prevented, and this finally leads to a reduced corrosion rate, as observed by Jen and Ostertag [21] and Miyazato and Hiraishi [22].

Figure 2. Photo on the left: Polyvinyl Alcohol (PVA) fibres [17]. Photo in the centre: High Modulus Polyethylene (HMPE) fibres [18]. Photo on the right: steel fibres [19]

The precise effect of fibres on the behaviour of concrete or SHCC is dependent on the type and volume fraction of the fibres used.The hydrophilic surface of PVA fibres is attributed to the presence of a hydroxyl group in their structure. They can retain their strength even at an elevated temperature of 150°C [23]. PVA fibres are well known for forming strong chemical bonds with the matrix, which require a considerable amount of energy to break [13]. However, the ductility of SHCC may be negatively impacted because the fibres may rupture before the bond slips. They have a reputation for high resistance to alkali, UV, chemicals, and abrasion, as reported by the manufacturer [16]. PVA fibres are also stable under heat and moisture exposure [24] and are not susceptible to corrosion. Nonetheless, their higher cost compared to other alternatives is a disadvantage.

The high modulus polyethylene (HMPE) fibres are long chains of aliphatic hydrocarbon [25], making it a thermoplastic substance with a glass transition temperature of approximately -120°C. These polymer chains are not vulnerable to chemical attack due to the absence of chemical groups that can attract acids, alkalis, or other chemicals at room temperature that could break the chains [26]. Consequently, HMPE fibres possess excellent chemical resistance. However, their hydrophobic nature causes them to form weak adhesion bonds with the cementitious matrix [13]. Despite their high tensile strength and modulus of elasticity, their low density keeps them relatively lightweight. Compared to PVA fibres, they are approximately twice as strong [13], but the ultimate strain at which they break is relatively low. Nevertheless, the weak matrix-fibre interface in combination with higher tensile strength leads to greater ductility in SHCC compared to SHCC with PVA fibres [27].

Steel fibres are commonly used due to their relatively low production costs. They marginally improve compressive strength, but can enhance tensile strength up to 40%, according to [28]. The use of steel fibres also results in significant improvements in post-peak ultimate strain and ductility. The bonding strength between steel fibres and the matrix can easily be adjusted compared to other types of fibres. Steel fibres with hooks can be manufactured to create mechanical interlocking with the matrix in addition to friction and adhesion at the interface [29]. However, steel fibres have disadvantages such as high self-weight and poor workability [28]. Steel fibres embedded in concrete are prone to corrosion when exposed to low-pH environments or chemicals [30]. Nevertheless, if concrete is well-compacted and sufficient cover is applied, this is not a threat to members' integrity and performance. The damage will be limited to the exposed surfaces.

3 Characteristic properties of SHCC

3.1 Shrinkage

PVA fibres and the fibre-matrix interface do not contribute significantly to the driving mechanism of, e.g., drying shrinkage, which is caused by moisture migration to a lower relative humidity environment. However, there are fibres that do absorb a significant amount of moisture, e.g., natural fibres. According to [31], natural fibres will swell or shrink depending on relative humidity. This change in strain has an influence on the bond between fibre and matrix.

The amount of drying shrinkage in SHCC is significantly higher when compared to that of NC [32]. The typical drying shrinkage of NC is in the range between 50 μm/m and 350 μm/m[33]. The ultimate drying shrinkage strain of SHCC may be between 1200 μm/m and 2500 μm/m [34, 35]. This means that the drying shrinkage of SHCC is at least twice as large compared to NC. The primary cause of such large drying shrinkage in SHCC is a relatively larger binder content in typical SHCC compared to normal (traditional) concrete. Since the shrinkage of aggregates is significantly lower than that of the hydrating paste, a material with a higher cement content will shrink more. The secondary cause is the small aggregate size that is used in SHCC. In general, large aggregates restrain shrinkage and reduce total shrinkage strain [36]. Still, as long as the strain capacity of SHCC is significantly larger than its shrinkage strain, it is expected that no localized cracks in SHCC will appear. Instead, it will have rather many fine shrinkage cracks [4].

3.2 Environmental burden of strain hardening cementitious composite

On the edge of climate change, no place on the globe would be free from the consequences of rising temperatures. The number of wildfires increases; sea levels are rising; contamination of drinking water is increasing; and the rate of decrease in biodiversity has never been faster due to human activity. Those events may seem overwhelming at first, but there are things that our civilization can do to prevent them from happening. From the construction point of view, if a structure has a longer service life, i.e., is more durable but at the same time more sustainable, this could be one of the main pillars of preventing global climate change from happening. In contrast to the past, the modern civil engineer is more aware of problems that the traditional linear building process may lead to.

Among all construction materials, concrete is currently the most widely used material on the planet. Unfortunately, concrete requires huge volumes of primary resources, which causes the depletion of natural resources. The production of cement is a highly energy-intensive process because the cement kiln has to operate at a high temperature of approximately 1450 to 1600°C. To sustain this heat energy, a large volume of fossil fuel is burned, leading to the release of pollution into the air, water, and soil. The largest problem is the emission of huge quantities of carbon dioxide (CO_2) into the atmosphere due to the burning of fossil fuels. CO_2 is released during the decomposition of calcium carbonate ($CaCO_3$) into calcium oxide (CaO) and CO_2. The environmental impact of the production of concrete may seem bad, but well-designed concrete members can last for many decades, even in a harsh environment. So, the environmental burden can be spread over many years of its service life. As a result, concrete may be a sustainable alternative if it is applied wisely.

The environmental impact of SHCC is higher than traditional concrete for three reasons. Firstly, SHCC makes use of fibres that have to be produced and transported, and are more difficult to recycle afterwards from the mix. This component material is not used in NC. Secondly, the lack of coarse aggregates in SHCC leads to a higher consumption of binder compared to NC. Thirdly, it is the larger portion of chemical admixtures and a super-plasticizer that have to be added, which also contribute significantly to the environmental impact of SHCC. To systematically analyse the environmental burden of both materials, Li [7] has conducted a Life Cycle Analysis (LCA) of SHCC and NC. The main results from this paper are shown in Figure 3. The functional unit in this paper was defined as ‘1000 kg of material’. It is a subject for debate if this is an appropriate functional unit. Due to the superior properties of SHCC compared to conventional concrete, less SHCC might be needed for the same performance criteria. For example, the application of SHCC link slabs is common in Japan. In a case study, for a life cycle of 60 years, a traditional bridge with conventional steel expansion joints was compared to a traditional bridge with SHCC link slab [37]. On a material basis, the production of SHCC consumes 1.8 times the energy consumed for the production of conventional steel-reinforced concrete (1% steel by volume). A similar trend is obtained if other sustainability indicators are compared. However, SHCC properties are expected to extend the service life of the SHCC system to twice that of the conventional system, resulting in significantly lower total life cycle energy consumption. Finally, the results indicate that the SHCC bridge deck system has 40% less life cycle energy consumption, 50% less solid waste generation, and 38% less raw material consumption. Construction-related traffic congestion and maintenance are the greatest contributors in most life cycle impact categories. However, it has to be highlighted that this analysis was based on the assumption that the SHCC link slab would double the life expectancy of the bridge deck relative to the conventional steel joint.

There are ways to improve the sustainability of SHCC. The development of green SHCCs, with examples in the adoption of alternative binder/filler, sand, and fibre, is analysed in [7]. Alternative ingredients may have a lower energy/carbon intensity, be sourced from industrial waste streams, or be renewable. According to [25], HMPE fibres have a lower carbon footprint compared to steel fibres or any other synthetic fibre (e.g., PVA fibres) due to the higher strength/weight ratio of HMPE fibres. However, SHCC sustainability derives mainly from its durability under a variety of exposure conditions, in particular the intrinsically tight crack width (below 100 μm) which minimizes the impacts of crack-related deterioration mechanisms. Apart from the tight crack width that slows the ingress of aggressive agents through the concrete cover, the ductility of SHCC provides an additional means of service life extension through the suppression of cover spalling tendency once reinforcement corrosion is initiated [7]. Therefore, care should be taken when introducing certain types of industrial waste or recycled sand into SHCC, as they could eventually affect the mechanical performance of SHCC and thereby its long-term benefits.

Finally, due to its relatively high environmental impact per unit volume, SHCC should not be used in places where its excellent durability aspects cannot be utilized. Instead, SHCC should be used as a durability enhancement for reinforced concrete at the most susceptible locations in the structure (e.g., cover, heavily loaded tension zones).

3.3 Bonding properties of Strain Hardening Cementitious Composite to concrete

When used for the repair of old/deteriorated concrete structures, traditional concrete or mortar is brittle and can exhibit large cracks or debonding of the interface. To improve the service performance of concrete structures and address the inherent brittleness of repair materials, SHCC was introduced as a promising repair material. For this reason, so far, research has focused mostly on the bonding properties between the freshly cast SHCC and the existing, old NC. The effect of the interface between freshly cast NC and an older SHCC, which might be a governing situation for innovative hybrid SHCC structures when SHCC is used as a stay-in-place mould for concrete, is rarely studied.

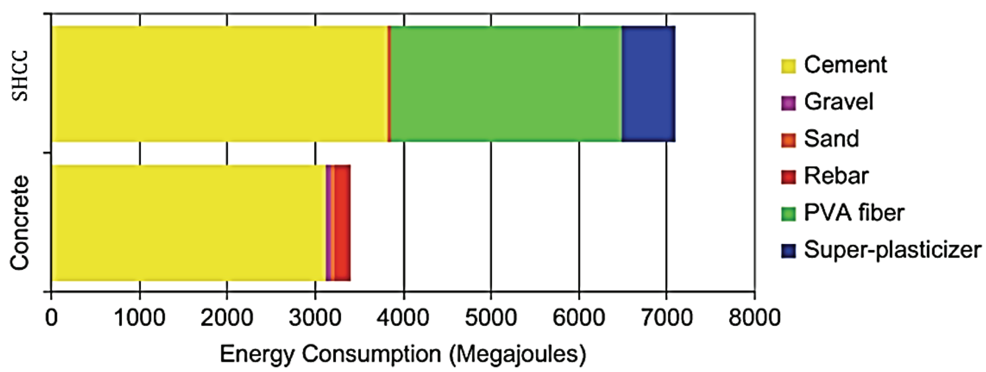


Figure 3. Energy consumption per 1000 kg of steel reinforced concrete and SHCC [7]

The bond strength of the SHCC-to-concrete interface under shear load depends on the compressive strengths of SHCC and NC, the curing age of the specimen, the curing environment (temperature and relative humidity), the interface roughness, the type of fibres used, and, if applied, additional binding agent strength. Those influencing factors were experimentally investigated by Tian et al. [38] and Gao et al. [39].

The research conducted by Tian et al. [38] has found that the roughness of the interface is the most dominant factor in the failure mode (as the higher roughness of a surface provides a larger contact area), while SHCC strength class and fibre types play a secondary role. The higher roughness of a surface provides a higher contact area. To determine the roughness of the surface, the surface profile of exposed aggregates on the interface was measured, and then the average height (the interface roughness value) was evaluated. According to [40], the limit value for the interface roughness value is about 4-5 mm. Values higher than this result in a weaker interface. In those experiments, the bond strength between cast-in-situ Ultra-High Toughness Cementitious Composite (UHTCC) and old concrete specimens was tested in a pull-out setup. UHTCC is similar to SHCC but has much higher strength. Furthermore, in [36], it was found that higher SHCC compressive strength led to higher interface shear strength and that PVA fibres with higher ultimate tensile strength only marginally influenced the shear strength [38].

Two types of specimens were widely used to investigate the shear bonding strength of the interface (Figure 4). Three slant shear specimens consisting of SHCC with a 28-day compressive strength of 39,9 (±0,38) MPa and NC C35/45 have been tested in the research presented by Gao et al. [39]. The mean shear strength value that has been found at room temperature is 5,5 MPa. However, two of the three specimens were broken before loading, so there is no data about the standard deviation or the variation coefficient. Furthermore, the heat treatment up to 200°C after standard curing for 28 days had a beneficial effect on the strength. The shear strength value at 200°C equals 6,87 (±0,87) MPa with a variation coefficient of 12,7%. The shear strength values after exposure to temperatures beyond 200°C were worse than at room temperature.

The researchers, Tian et al. [38] used single-sided shear specimens to obtain shear strength. In this research, one type of NC C40/50 and four different types of SHCC with different 28-day compressive strengths (from 21.7 MPa up to 40.8 MPa) have been used. The interface shear strength was found in a range between 0,33 (±0,04) MPa for low-strength class ECC and 1,11 (±0,15) MPa for high-strength class SHCC. The specimens with a thick epoxy resin layer with coarse aggregates applied on the interface resulted in the following shear strength range: 0,86 (±0.08) MPa for low-strength SHCC and 3,33 (±0,13) MPa for high-strength SHCC.

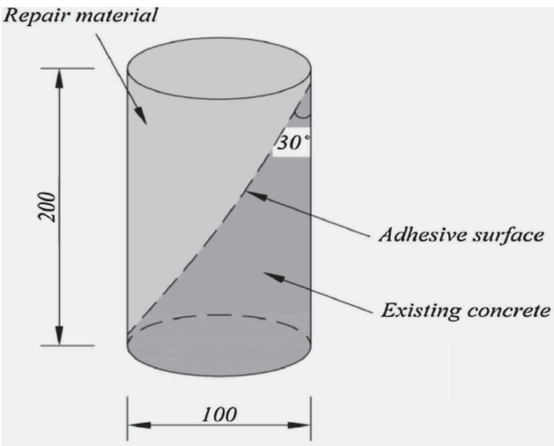
To enhance the bonding strength of the interface, additives could be added at the cement manufacturing stage. According to relevant literature [40, 41], fly ash, slag, and silica fume can improve the bond properties of the interface. In addition, there are different admixtures that could improve bonding property, e.g. expansive agent and SBR latex [40]. According to the slant shear test conducted by [41], 52,8% higher bonding strength was obtained by SHCC with slag at the age of 28 days compared to the monolithic concrete reference specimens. SHCC with fly ash improved the bonding strength by 36,4% compared to the reference specimens. The reference specimens were made of concrete with a 28-day compressive strength of 31,9 (±1,1) MPa.

The results presented in [24, 41, 42] show that concrete with PVA fibres had significantly better bonding performance than NC, so in general, the SHCC-to-concrete interface was stronger than a concrete-to-concrete interface.

The main conclusion drawn from these experiments is that SHCC can achieve a strong bond with concrete. Moreover, this should be possible without any prior preparation of the surface. This is a rather promising clue for further research, and even more so for practical applications where practices such as preparation are most likely very costly and environmentally expensive. Although a thick epoxy resin layer with coarse aggregates gives the highest interface shear strength, it is unlikely to be used in practice due to its high cost. Nevertheless, it can increase strength by a factor of 3, as demonstrated in [38].

3.4 Shrinkage induced debonding

The differential (drying and/or thermal) shrinkage between SHCC laminate and NC may cause a bonding failure at the interface. Restrained drying, shrinkage degradation, and resulting interface stresses are the major contributors to this failure [4]. Due to restrained shrinkage deformations, the generated stress causes delamination between SHCC and NC. Therefore, the properties of SHCC should be chosen not only based on strength performance but also on the exposure environment and ductility of the interface.



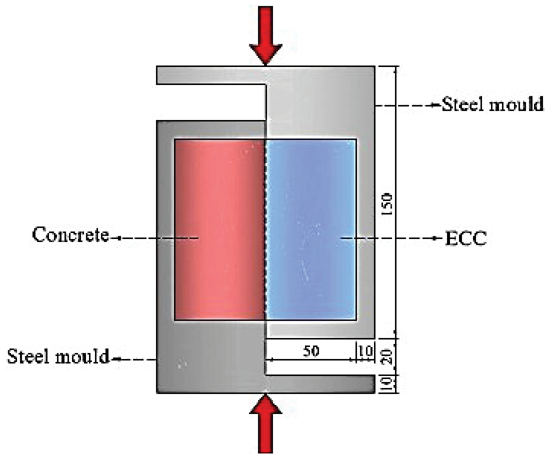


Figure 4. Left: The slant shear specimen according to American ASTM C882 standard [39]. Right: Single-sided shear test setup [38]

Next to material properties, an interlocking mechanism plays a significant role in counteracting the shrinkage at the interface between SHCC and NC. Despite reviewing the database of scientific papers and books, no systematic experiment has been found that evaluated the effectiveness of SHCC surface roughness on damage caused by shrinkage and its residual interface strength .

4 Experimental benchmarks on RC beams with shear strengthening using SHCC

The major advantages and disadvantages of SHCC have been described above. In theory, an optimal solution would be to apply SHCC and NC together in a so-called hybrid system. In this way, those materials should cover their mutual shortcomings. This chapter will give an overview of the experiments in which hybrid SHCC-concrete beams were tested under shear loads.

Before 2015, there was hardly any knowledge about the shear behaviour of hybrid SHCC-concrete beams. The first research on the shear behaviour of RC beams without transverse reinforcement strengthened by SHCC layers was conducted by Zhang et al. [43]. In 2019, Wang et al. [44] conducted a similar experiment but on slightly larger members and thicker SHCC layers. A year later, Wei et al. [45] published their work on the shear behaviour of hybrid RC beams with transverse reinforcement. The experiment was successful, but their hybrid beams experienced minor delamination of SHCC laminates just before the peak load.

There are also some other types of hybrid beams strengthened by SHCC and tested for shear capacity. In 2018, Wu et al. [46] tested RC beams strengthened by precast thin-walled (20-mm) U-shape UHTCC. Multiple M16 penetrating bolts have been added to improve the integration of the U-shape. The increase in shear strength reached 67,4% [46]. In 2020, Shang et al. [47] proved that U-shape SHCC with stirrups is an effective way of shear-strengthening damaged RC beams due to fire. Recent research (2022) by Li et al. [48] showed the great potential of thin-walled (15-mm and 25-mm) U-shapes in their experiments to enhance the shear strength of RC beams with and without transverse reinforcement. The relative increase in shear resistance ranged between 8,40% and 66,39% [48]. Two relevant studies, considering SHCC strengthening of a beam with and without shear reinforcement, are further presented in detail.

Experimental investigation by Zhang et al. [43] on shear capacity of RC beams without transverse reinforcement strengthened by SHCC laminates

The paper presents an experimental investigation of the SHCC laminate-strengthened RC beams without transverse reinforcement. This research focuses on the shear load carrying capacity of such a hybrid beam. Additionally, they documented the crack pattern of their hybrid beams.

In Table 1, the list of ingredients was provided for SHCC, with a 28-day compressive strength of 91 MPa used during the experiment [43]. As it can be deducted, the water-cement ratio equals 0,27, and the water-to-binder (cement + silica fume) ratio equals 0,22. The results obtained from the uniaxial tensile test on the dog bone specimens are shown in Figure 5. All specimens exhibited significant strain hardening behavior until ultimate tensile strength (point B1 or B2). Multiple fine cracks occurred and propagated after reaching the initial cracking (point A) until reaching the peak strength. Thereafter, tensile stress decreased due to the localization of some cracks. Young's modulus of SHCC is estimated to be around 29 GPa.

Table 1. Mix proportions of SHCC[43]

Component	Dry Weight [kg/m^3]
Cement {not specified}	1267,9
Silica fume	230,8
Fine sand	153,9
Expansion agent	40,0
Water	338,5
Superplasticizer	15,4
PE fibres	14,6
Air reducing agent	0,06

The list of ingredients in NC was not provided in this paper. The only information known about this concrete is that it had a 28-day compressive strength of 27 MPa and a Young's modulus of 23,5 GPa.

In Figure 6, the schematization of the hybrid beam is shown. The specimens were reinforced with two steel longitudinal ribbed bars with a diameter of 10 mm. No shear reinforcement has been applied. The steel, which was used for this reinforcement, has a yield strength of 345 MPa and a Young's modulus of 200 GPa [43]. The beam span was chosen to be 1 m, which results in 0.5 m of shear span, and

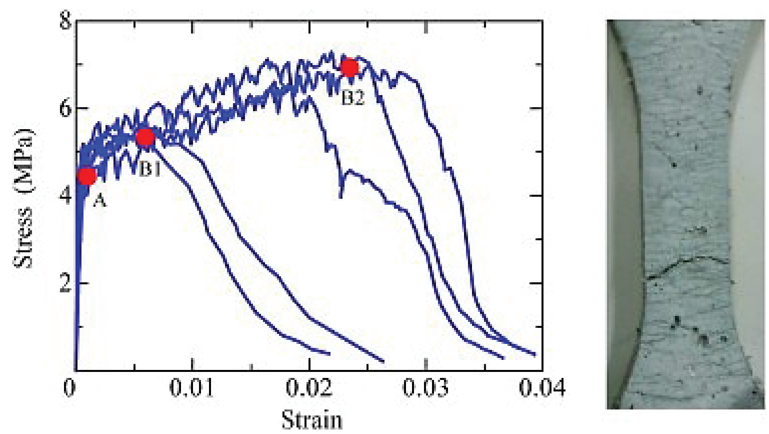


Figure 5. Uniaxial tensile test results of SHCC [43]

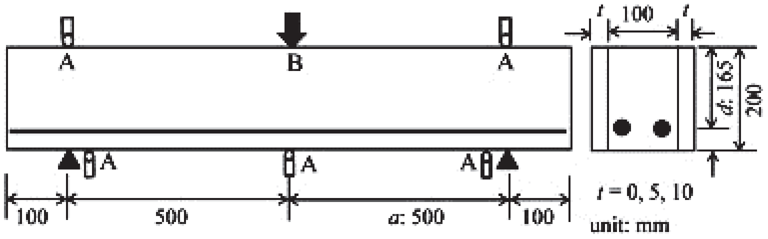


Figure 6. Geometry of RC beams strengthened by SHCC laminates [43]

this corresponds to $\eta_a \approx 3{,}0$. The beam, having a cross-section of 100 x 200 mm^2 was strengthened by casting SHCC laminates with a thickness of 5 mm or 10 mm on two sides. Before SHCC was cast on the side surface, those sides 'were washed out using a retarder to obtain roughed surfaces' quoting from [43].

Figure 7 shows the results of the experiment. The beam strengthened by 10 mm SHCC laminates has reached the highest shear load capacity of about 90 kN. That is almost twice the capacity of the reference beam. The beam strengthened by 5 mm SHCC laminates has reached about 70 kN. Even though Young's modulus of this SHCC is higher than that of normal concrete, the beams followed the same linear elastic branch up to a certain point.

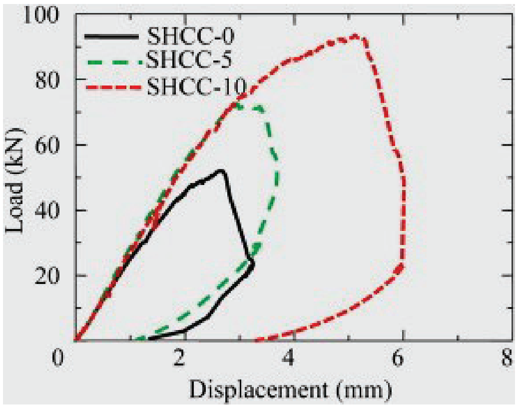


Figure 7. Load–displacement curve. SHCC-0 is the RC beam without SHCC laminates. SHCC-5 is the RC beam with 5 mm thick SHCC laminates. SHCC-10 is the RC beam with 10 mm thick SHCC laminates [43]

A comparison between the ultimate crack distribution of the shear-failed SHCC member and structural elements strengthened with the SHCC layer is demonstrated in Figure 8. For the SHCC member, it was observed that there were many multiple fine cracks in the diagonal shear direction of the SHCC member due to the fibre bridging effect. The beam finally failed in shear due to the localization of a critical crack. Finally, there was only one localized diagonal shear crack with a few accompanied small cracks in the vicinity, indicating that the ductility of SHCC has not been fully exploited when used for shear strengthening of RC members. This is in line with earlier observations when SHCC is used for repair.

Experimental investigation by Wei et al. [45] on shear capacity of RC beams with transverse reinforcement strengthened by high-strength SHCC laminates

This experimental study has a more realistic scenario than the previous one due to the use of transverse reinforcement. This research tries to answer the following question: whether SHCC laminates are efficient in the shear strengthening of reinforced concrete structures?

Table 2 Mix proportions of HS-SHCC[45]

Component	Massa ratio
Cement {not specified}	0,8
Silica fume	0,2
Sand	0,3
Water	0,2

+ 2% PE fibres by volume of the mixture

2 provides the mix design list for the HS-SHCC used. To maintain workability, a polycarboxylate-based superplasticizer was added to the mix. Polyethylene (PE) fiber (12mm long and 24µm in diameter) was chosen due to its excellent tensile strength and high modulus. Very fine sand with particle sizes of 0.125mm - 0.18mm was used.

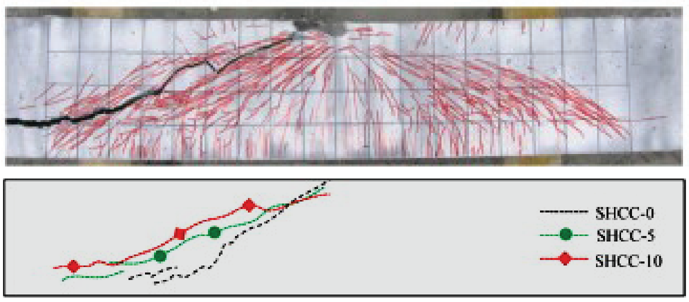


Figure 8. Positions of localized cracks. SHCC-0 is the RC beam without SHCC laminates. SHCC-5 and SHCC-10 are the RC beams with 5 mm and 10 mm thick SHCC laminates, respectively [43]

According to tests on small cubes (40x40x40 mm^3) by Wei et al. [45], a 28-day compressive strength of 120 MPa has been reached. Furthermore, the tensile strength of 10 MPa on dog bone specimens has been reported, as seen in Figure 9, and Young's modulus of 35 GPa.

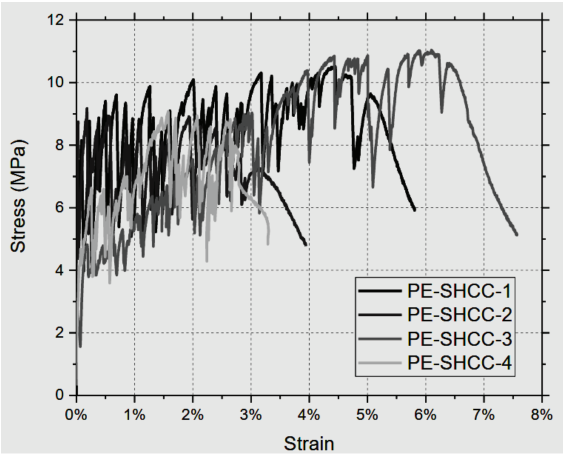


Figure 9. Tensile stress-strain curves of HS-SHCC 28 day direct tension test [45]

The properties of the NC were as follows: 36 MPa for 28-day compressive strength, and 26 GPa for Young's modulus. The compressive strength was tested on 100 x 100 x 100 mm^3 cubes.

This research paper [5] documents experimental beams with two different shear span parameters: 'Group A' with $\eta_a = 1{,}5$ and 'Group B' with $\eta_a = 2{,}5$. In group B, four beams have been tested: two reference beams and two hybrid beams. The detailed geometry of the beams is shown in Figure 10. The hybrid beams were only strengthened on one side (in the red area). The reinforcement steel has a yield strength of 585 MPa and a Young's modulus of 200 GPa. The SHCC laminates had a thickness of 10 mm and were cured for 28 days. Those laminates were cast directly on the surfaces of the beams. The loading speed for all beams was set to 0,01 mm/s.

All beams have failed in shear and developed large diagonal cracks. Furthermore, minor debonding of SHCC laminates was initiated, but they did not completely delaminate from the beams. In Figure 11, the results of the experiment are shown.

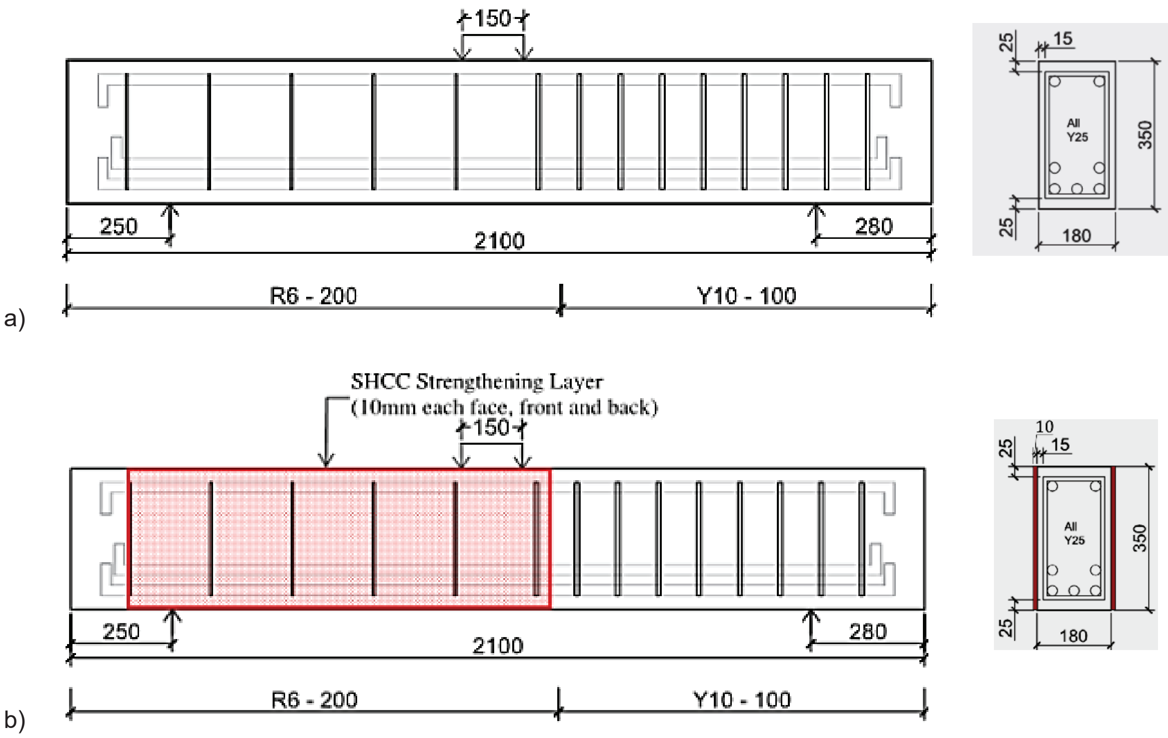


Figure 10. Group B: a) Geometry of reference RC beams. b) Geometry of hybrid beams strengthen by HS-SHCC laminates (red area) [45]

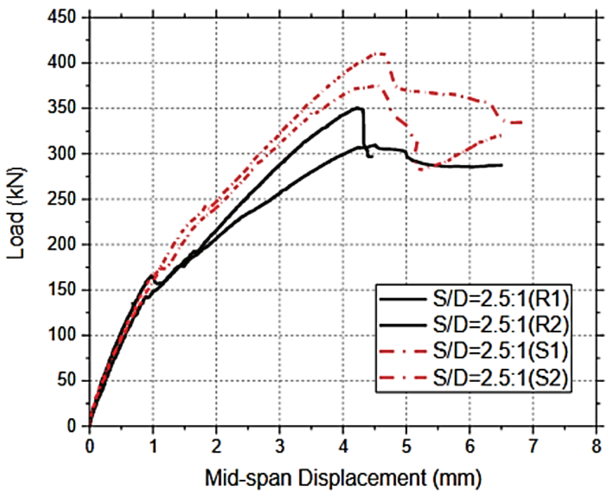


Figure 11. Load–displacement curve. R1 and R2 are the refence beams.
S1 and S2 are hybrid beams [45]

The shear capacity of the hybrid beam has increased by 19% compared to the reference beams [45]. The strength of the interface between NC and HS-SHCC was what determined the strength of the hybrid beams. This could mean that the utilization of HS-SHCC laminates was not complete, and thus the hybrid beams could have reached a higher shear capacity than the results presented in Figure 11. The shear failures of the hybrid beams were still of a brittle nature, like the shear failures of the reference beams.

5 Conclusions

Based on the above presented literature review, the following conclusions can be drawn:

- SHCC has superior tensile properties compared to concrete. Those properties are highly dependent on the composition of the mix:
  - The main benefit of SHCC is its high ductility. The range of tensile strain at 90% strength is somewhere between 2% and 5% [18, 43].
  - A typical crack pattern in SHCC consists of multiple parallel cracks. This is more advantageous than one concentrated crack like in NC because the width of an individual SHCC crack is significantly smaller. The width of cracks in SHCC ranges between 60 μm and 100 μm. The advantages of a smaller crack width are:
    - Good self-healing properties,
    - Smaller water permeability and ingress of hazardous substances: water with ions is one of the ingredients that lead to the corrosion of reinforcement.
  - The common range of the tensile strength of an SHCC is between 2 and 8 MPa. Yet, it highly depends on many factors like type of binder, w/c ratio, fibre volume fraction, and type of fibres. High-strength strain-hardening cementitious composites (HS-SHCC) with over 10 MPa tensile strength [43] have also been developed.
- SHCC has great durability but does not belong to the low environmental burden materials, according to Li [7]. SHCC can be more sustainable than NC only if its superior properties are utilized. In other cases, there is more damage done to the natural environment than is worth it.
- There is still a lack of knowledge regarding the concrete connection between old SHCC and young NC. Most of the current experiments [43, 44, 46] on this subject have been performed on the interface between young SHCC and old normal concrete. Based on experiments [38] conducted by Tian et al., the positive effect on the interfacial shear strength is mainly due to higher SHCC compressive strength and interfacial roughness. The secondary parameter, which is positively correlated with the interfacial shear strength, is the ultimate tensile strength of fibres, according to data presented in [38]. Based on experiments [39] conducted by Gao et al., the limited temperature treatment (< 200°C) might be beneficial to the bonding performance of the interface, but at extreme values (> 200°C), the interfacial shear strength is lower. Furthermore, Şahmaran et al. [41] have discovered that SHCC with slag has a higher bond shear strength than SHCC with fly ash. However, the contribution of slag in SHCC should be denoted as the secondary parameter since the primary parameters (SHCC compressive strength and interfacial roughness) had much greater effects on the interfacial shear strength.

- The most effective way to increase interfacial strength between SHCC and NC is to add roughness to the surface and increase the strength of SHCC. This can be used as guidance when designing the hybrid interface. So far, no efforts have been made to increase the ductility of the interface.

- The biggest disadvantage of SHCC is its significant magnitude and rate of drying shrinkage compared to that of NC. In most cases, the drying shrinkage of SHCC is at least twice as high as that of NC. This has a huge negative consequence for interfaces between SHCC and NC because they are prone to delamination.

- Recently, scientists conducted a few experimental investigations [43-48] on the shear behaviour of RC beams, with and without transverse reinforcement, strengthened with SHCC laminate. Using SHCC laminates with a thickness of 10 mm improves shear capacity by 18% to 50%. This ratio is dependent on the tensile and elastic properties of SHCC compared to those of the base concrete. The shear failures of those hybrid beams were still as brittle and sudden as those in control groups. Some of the tested hybrid beams showed debonding of SHCC laminates, which resulted in premature failure.

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Review paper

The supply and demand of infrastructure robustness, resilience and sustainability

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resilience,
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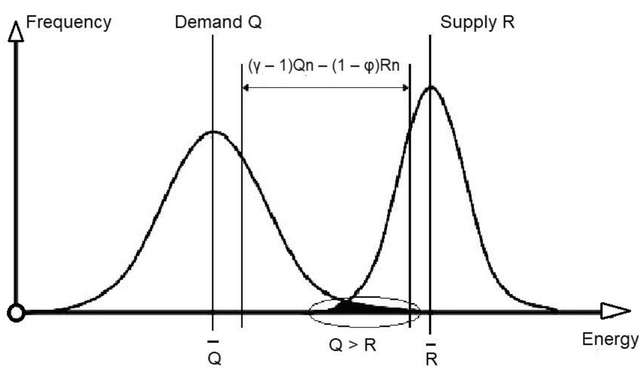
ABSTRACT

Economics and engineering manage the built infrastructure in a dynamic equilibrium of supply and demand. The potentially contradictory constraints of the two domains within their governing dimensions of money and energy are explored. An analogy is drawn between mechanical and infrastructure network stability. The recently popularized terms of robustness, resilience and sustainability are defined as critical descriptors of the process and product to be jointly optimized by the two fields.

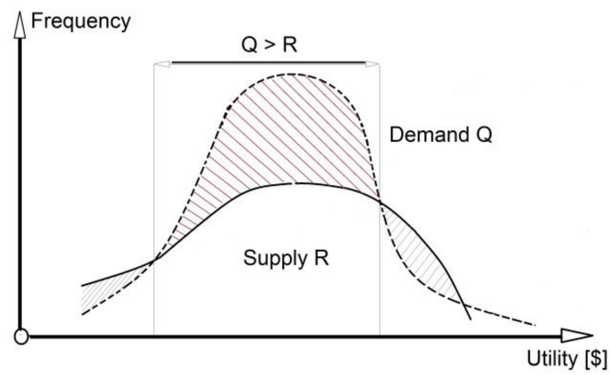
1 Introduction

In the present view, economics and engineering manage infrastructure processes and products as supply / demand (S/D) relationships in terms of money and energy. Figs. 1 – a) and b) illustrate the contrasting priorities governing engineering and economics. Energy is viewed as a rigid constraint, whereas money is regarded as a negotiable restraint. Thus, infrastructure management must reconcile the supply of and demand for structural performance under physicaly rigid constraints dimensioned in energy and economically negotiable restraints, negotiated in a dynamic mix of ultimately monetized economic and political priorities.

Both engineering and economics balance supply (R) and demand (Q), however their respective restraints and constraints can appear diverging. Engineered products must supply performance exceeding service demands by prescribed and uniformly accepted factors (such as γ and φ in Fig. 1. a) over an intended useful life. In contrast, economic processes are planned over strategically and tactically varied time horizons. Except in the extreme high - and low - income areas, where social programs and philanthropy may reverse the governing pattern of Fig. 1. b), service demands exceed the supply by an indeterminate degree and motivate social progress. These diverging constraints and restraints are expressed in Eq. 1 – a, – b, as follows:



Figue 1. (a) The engineering constraint



(b) The economic restraint

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Engineering products: $R > Q$ [Energy] (1–a)

Economic processes: $R < Q$ [$] (1–b)

Violating either condition of Eq. 1 amounts to failure. Less obvious, harder to address and more common are the failures of the two fields to reconcile the ostensibly contradictory constraints of their incongruent models and to render them compatible.

Engineering ensures a ‘stable’ equilibrium, such that $R>Q$ in terms of energy. ‘Conservative’ oversupplies are professionally established and legally enforceable. Economics negotiates a ‘dynamic’ equilibrium, such that $R<Q$ to an indeterminate degree in terms of money. ‘Shortfalls’, even catastrophic ones, are customary. Subject to litigation can be shortages in engineering and excesses in economics. Engineering products are acquired ‘ground-up’ under natural constraints in response to top-down economic demands. Economic processes are transacted ‘top-down’ under fiscal restraints in response to ground-up social demand. Hence, economists tend to regard engineering products as ‘static’, whereas engineers tend to view economic processes as ‘unstable’. Few if any are expert in both domains. The proverbial ‘meeting in the middle’ implies unattainable perfection, occasionally promised in political campaigns. Hence, both engineers and economists regard with skepticism up-to-the-moment politics, a.k.a. ‘the art of the possible’. Although only implicit in Fig. 1, politics dominates infrastructure management in the (also implicit) domain of intelligence / information.

Bridges are critical links in the built infrastructure, supplying instructive examples of Q/R disparities dimensioned in money and energy. The present exercise expands from events in the bridge network of a major city to more general conclusions applicable to infrastructure management in general. The first step is to examine the engineering methods of assessing the supply of structural performance.

2 Bridge conditions

The Federal-Aid Highway Act of 1968 initiated modern vehicular bridge management in the United States, and by extension, worldwide. The National Bridge Inventory (NBI), established by the Federal Highway Administration (FHWA) rapidly built a database of 230,000 bridges, eventually expanding it to nearly 650,000. A vehicular tunnel database was initiated in 2015. Integration of the railroad bridge database, exceeding 220,000 bridges is pending.

In its present form, NBI is equipped to support strategic lifecycle decisions on local and national levels. Originally however, its overwhelming priority was to identify and avert disasters, such as the collapse of the Silver Bridge at Point Pleasant in 1967. Tactically, potential hazards had to be promptly identified and mitigated. Strategically, realistic life-cycle bridge performance had to be modeled, anticipated, and optimized. To these ends, the Act [1] mandated biennial inspections of vehicular bridges. To serve both objectives, the visual biennial inspections had to supply actionable qualitative and quantitative assessments of bridge conditions.

The NBI compensates for the vagueness of the term ‘condition’ with a database of complementary qualitative and quantitative, descriptive and prescriptive bridge assessments. Local owners supplement NBI according to their specific needs. The resulting condition database supports bridge management decisions on both project and network levels. Milestones in that process were the introduction of the LRFD Bridge Design Specifications by the American Association of State Highway Transportation Officials [2] and the AASHTO Bridge Element Condition States, adopted in [3].

The biennial inspections update the NBI with two types of assessments: descriptive and prescriptive. The original 10 – level condition ratings were essentially descriptive. The 4 element level condition states which superseded them combine the descriptive opinions of qualified engineers with quantitative measurements and, at the lowest level 4, imply prescriptive recommendations. Prescriptive assessments recommend action. Such are the ‘flag’ reports of potential hazards according to New York State Department of Transportation (NYS DOT) defined in [4, 5]. Based on its bridge inventory, NYS DOT also recognizes a number of vulnerabilities, such as steel details, concrete details, seismic, hydraulic, collision, overload, and acts of destruction. The vulnerability of overload was withdrawn. The variety of the federal and NYS DOT bridge condition assessments is summarized in Table 1.

Table 1. Bridge assessments

Assessment	Type	Source	Description
Element condition ratings	Descriptive	[1]	9 (New) – 0 (Imminent failure)
Bridge serviceability appraisal	Descriptive	[1]	9 (Superior to design criteria) – 0 (Closed)
Maintenance ratings	Prescriptive	[6]	9 (No repairs needed) – 1 (Closed)
Sufficiency ratings	Computed by weighted formula	[1]	0 < S1 + S2 + S3 – S4 < 100%, where: S1 – Structural adequacy & safety (< 55%) S2 – Serviceability & Obsolescence (< 30%) S3 – Essentiality for public use (< 15%) S4 – Special reductions (< 13%)
Load ratings	Computed analytically	[1, 2]	Inventory & Operating ratings
Element condition states	Descriptive Prescriptive	[3]	4 (Good), 3 (Fair), 2 (Poor), 1 (Severe)
Element condition ratings	Descriptive	[4]	7 (New) – 3 (Not functioning as designed) – – 1 (Totally deteriorated or failed)
Potential hazards (Flags)	Prescriptive	[4]	Structural (PIA, Red, Yellow), Safety
Vulnerabilities	Descriptive Prescriptive	[5]	Hydraulic, Steel, Concrete details, Collision, Seismic, Destruction, Overload (withdrawn)

In another significant development, advanced technologies are offering a variety of non-destructive testing and evaluation (NDT & E) techniques [6], allowing for a quantification of previously purely qualitative assessments.

The qualitative condition ratings and quantitative diagnostics describe 'as is' conditions on the 'project' or 'ground-up' level. Also ground-up (a.k.a. hands-on), the prescriptive flag reports identify potential hazards, requiring a timely resolution. Load ratings, flag resolutions, and vulnerabilities are determined at the 'top-down' network level.

Serviceability combines ground-up findings and top-down determinations. The bridge management database integrates the overlapping complementary assessments in a 'bilateral' flow between the project and network levels. As in a redundant mechanical structure, the strengths of one block of information compensate for the weaknesses of another, enabling the redistribution in the event of partial failure. In subsequent sections, serviceability is qualified and to a degree, quantified, in terms of robustness, resilience and sustainability.

Up to 2015 bridge inspections according to the several updates of [4] included the following significant features:

– Inspection team leaders are professional engineers licensed in N.Y. All inspectors pass a state course;

– Fracture-critical elements are inspected hands-on and certified by the team leader;

– All bridge elements were rated in all spans on a scale from 7 (new) to 1 (failed), 3 signifying 'not functioning as designed';

– Potential hazards are designated as flags and processed in advance of the inspection reports.

In 2016 [5] adopted the four element condition states recommended by [2], superseding the seven condition rating levels of [4]. The other features pertain.

In their incongruous dimensions, the various assessments supply a multi-faceted view of the infrastructure and of each other. The 'bridge condition' and 'sufficiency' ratings of the 790 vehicular and pedestrian bridges of New York City, enumerated in Table 2, are plotted in Fig. 2 for 2008.

Table 2. New York City bridges & tunnels

Type	Quantity
East River Crossings	4
Moveable	25
Waterway	51
Arterial	208
Off – system (Local)	389
Pedestrian	107
Tunnels	6
Total	790

The two sets of ratings plotted in Fig. 2 are obtained by different weighted average formulae and hence, are fundamentally qualitative. The former is based on the NYS DOT descriptive condition ratings (7 – 1) according to [4]. The latter is based on the FHWA ratings (9 – 0) [1], comprising assessments of importance, serviceability, and obsolescence. The data points of both sets are generated deterministically, however in their continually expanding aggregate they offer abundant material for statistical, frequentist, and other probabilistic interpretations. Consistently with the basic management commitment to safety, structural conditions rated ≤ 3, i.e., not functioning as designed according to [4] are few. In contrast, the FHWA sufficiency ratings < 50% according to [1] are numerous. The conspicuous outliers in both graphs reflect rehabilitations. There are no outright structural failures, but quite a few serviceability ones. If the two sets of data points were reduced to average patterns over time, the 'structural condition' graph would be concave, tending asymptotically towards an average rating of 4, whereas the 'sufficiency rating' one would be convex, declining to 0 at about 85 years (essentially consistent with the 75 yeas useful life recommended by [2] and earlier editions. As postulated in Fig. 1 and Eq. 1, structural safety meets the demand, but serviceability is undersupplied. Both sets rate performance, however the 'condition rating' assesses structural integrity in engineering terms, whereas the 'sufficiency' rating reflects user's satisfaction, hence containing economic considerations. Reversing the trends would be unsafe in the former case and possibly unaffordable in the latter.

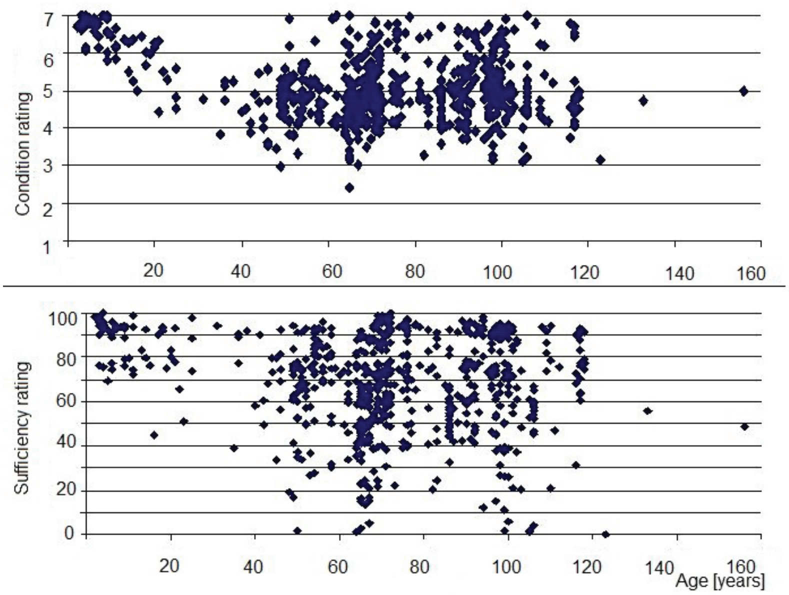


Figure 2. Structural condition and Sufficiency ratings for the NYC bridges (circa 2008)

The relationship of condition and load ratings similarly confirms the safe operation of the network. Qualitative visual inspections are the first to rate bridges unsafe, thus requiring AASHTO load ratings to determine whether the structure has quantifiably acceptable load-bearing capacity. The latter can be of levels I, II, and III, and may include proof loading. It is not uncommon for load ratings to find bridges deemed in fair to poor structural condition still fit to carry design loads. Once again, the reverse would have amounted to a misplaced relationship of the qualitative and quantitative assessments.

The average bridge life of 80 to 100 years suggested by the descriptive condition ratings is deceptive. For a realistic assessment, they must be combined with the prescriptive flag reports. The low – rated bridges of ages 40 to 45 years generate the most flags and govern the needs for repair and reconstruction. So long as deterioration is not delayed by other means, new bridges decline into this category while the current ones are rehabilitated. The NYS DOT flag protocol was designed as the first line of defense against the proliferating potentially hazardous bridge-related conditions and in the late 1980s became the critical descriptor of the state of the network. Flags are defined in [4, 5] as follows:

Red Flag - A structural flag that is used to report the failure or potential failure of a primary structural component that is likely to occur before the next scheduled biennial inspection.

Yellow Flag - A structural flag that is used to report a potentially hazardous structural condition which, if left unattended could become a clear and present danger before the next scheduled biennial inspection. This flag would also be used to report the actual or imminent failure of a non-critical structural component, where such failure may reduce the reserve capacity or redundancy of the bridge but would not result in a structural collapse.

Safety Flag - A flag that is used to report a condition presenting a clear and present danger to vehicular or pedestrian traffic but poses no danger of structural failure or collapse. Safety Flags can be issued on closed bridges whose condition presents a threat to vehicular or pedestrian traffic underneath or in their immediate vicinity.

Prompt Interim Action (PIA) – A flag demanding resolution by the responsible owner within 24 hours.

Defined as 'potential hazards', flags may or may not signify element or service failures. Their veracity and gravity can vary widely. The bridge management action they invariably require is engineering review.

Applied jointly to the NYC bridge network of the considered period, the described structural assessments reassure tactically and disturb strategically. The 'condition' and 'load' ratings indicate acceptable bridge safety. However, failing 'sufficiency ratings' and accumulating 'flags' may foretell a pending crisis. In the more recent terminology, discussed in the subsequent sections, even though the individual structures appear on the average robust, the network's resilience and overall sustainability may be approaching instability.

3 The NYC bridge network

During the last several decades, the vehicular bridges managed by NYC DOT have fluctuated around the numbers in Table 2. Without adjusting original dates of completion for rehabilitations, their average age circa 1990 was approximately 75 years. Another approximately 600 bridges on the arterial network in the five city boroughs are managed by NYS DOT. Their average age was approximately 40 years. Span numbers quantify bridge networks more meaningfully. NYC DOT manages approximately 5,000 spans.

Following the economic restrictions of the 1970s and early 1980s, the NY City bridge network suffered extreme neglect. By 1989 80 City bridges had been fully or partially closed and many were posted for restricted load. As intended, the proliferating flag incidence clearly signaled the unfolding network crisis. The flag history of the New York City bridges from their inception in 1982 to the 'steady state' reached circa 2006 is illustrated in Fig. 3 and discussed herein.

The following five periods are discernible in Fig. 3:

1982–1987 Apparent equilibrium following initial adjustments (A – B)

1987–1992 Increase reaching annual factor of 2 (B – C)

1992–1996 Peaking approximately 24 times above the initial level (C – D)

1996–1999 Annual decrease by a factor of approximately 1.24 (D – E)

1999–2006 Apparent equilibrium at approximately 10 times the initial level (E –).

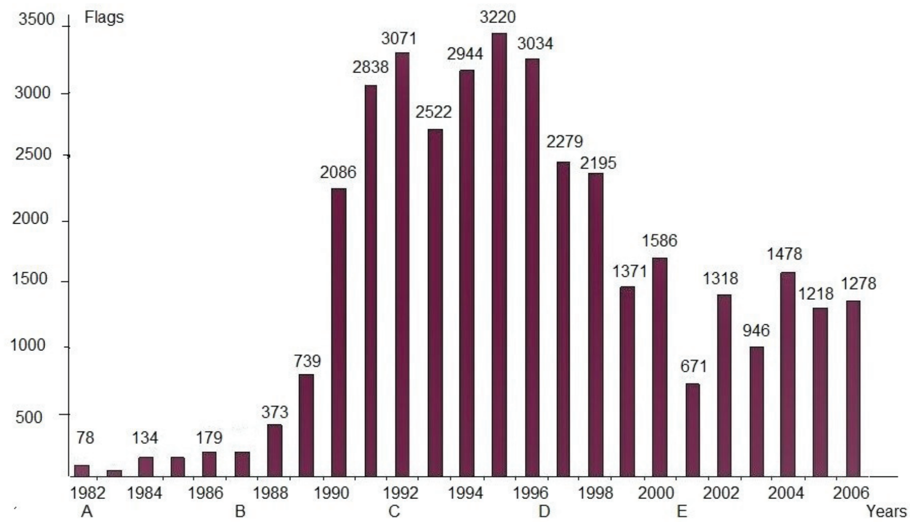


Figure 3. Flags on the New York City bridges, 1982 – 2006

Beyond 2006 the flag numbers have fluctuated about the number of 1200, suggesting a new equilibrium of service demand and bridge network performance supply.

During the years under consideration the direct costs of the (mostly temporary) repairs mitigating flaged conditions were averaging at approximately $US 15 - 20K. The rough estimates of the notoriously intractable user costs due to traffic interruptions are invariably higher. The costs of the potential hazards escalating to actual accidents can be vaguely estimated, based on annual court case settlements in New York City. As a result, all levels of city management recognized the urgent need to address the looming crisis in bridge conditions.

Two events particularly impressed the public attention. In 1988 bridge inspectors found the deterioration of the Williamsburg Bridge, crossing East River since 1903, so advanced that its eight vehicular lanes and two subway tracks were temporarily closed. Following an in-depth inspection and analysis, [7] concluded that a rehabilitation, at a cost exceeding $US 1 billion was feasible and urgent. On June 1, 1989, a piece of concrete spalled from the underside of the Franklin Delano Roosevelt (FDR) Drive on the Manhattan East Side at 19th St. and killed a motorist, as reported in [8].

A less visible, but no less significant consequence of the events was the re-establishment of the Bureau of Bridges (later Division) at the New York City Department of Transportation (NYC DOT). The financial crunches of the 20th century had reduced the powerful Bridge Commission of the early 1900s to a lesser department in various agencies for more 'general services'.

According to inspection reports nearly half of the City bridge decks were in conditions similar to FDR's. Under its constraining circumstances, the new City Bureau of Bridges had to obtain emergency funding and retain qualified in-house and contracted expertise. Hence, it needed a credible projection of the hazard mitigation needs. The Bridge Inspection & Management Unit established by the author undertook to model the flag expectations resulting from the next inspections. The first steps in that process for 1991 were reported in [9]. The projection for 1992 was reported in [10].

'Extreme events' of varying duration afflict the various aspects of social activities and assets at different frequencies. To varying degrees, they combine randomness and phenomenological causation. Thus, anticipating and managing any extreme event should benefit from both the random and causal features they may share. Traffic, climate and other factors cause structural deterioration by unrelated mechanisms, but in their domains correlate with population density, and human activities. Even without a full understanding of the underlying phenomena, network management plans for their identification and mitigation based on statistical data. During the ostensibly stable period A – B of Fig. 3 (1982 – 1987), however, frequentist reasoning alone misses the escalation during B – C (1987 – 1992). As [9] and [10] reported, causes for that development were discernible in the element 'condition' ratings.

The highly site / moment (i.e., space / time) – specific vehicular bridge network of New York City in the reviewed period qualify, retrospectively, as an extreme event gradually evolving from a 'potential' to an active crisis. A posteriori, the developments from 1987 to 2006 argue convincingly for prevention. However, the 'gestation' period between 1982 and 1987 could not have justified an emergency budget request, even though the five stages of the flag pattern are typical of most disaster scenarios and hence, could have been predictable.

4 Supply & demand of services and expenditures: a stability analogy

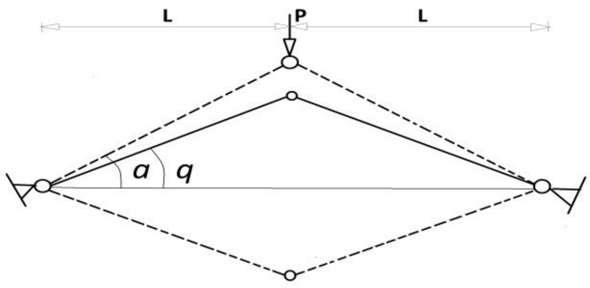
Adopting Ernest Hemingway's words describing a character's bankruptcy in The Sun Also Rises (1926), crises occur first gradually, then suddenly. According to [12] a 'crisis' is "a state of rupture, negative and instantaneous, along a *trend* or 'tendency'". In [13] Parrochia traces the evolving view of 'crises' in all spheres of social activity from antiquity to the present. He views them as events, discontinuities, conflicts, and transactions.

Many phenomena qualifiable as critical or catastrophic display 5-stage patterns similar to those of Fig. 3. Such are the financial and political so-called crises and the health epidemics, including COVID-19 in the United States during 2020 – 21. Also similar are the phases of the 'Future Tech Hype Cycle', consisting of innovation, expectations, trough of disillusionment, enlightenment, and plateau of productivity, and the stages of grief, comprising denial, anger, bargaining, depression, and acceptance. Given any specifics, the cycles comprise an apparent equilibrium of supply and demand, imperceptibly degenerating from stable to neutral and to unstable, dynamic change perceived as collapse, peaking (or "hitting bottom"), and attenuation to a stable new equilibrium at elevated supply and demand. The moments when this scenario can be averted, for example by a smooth transition from the initial to the final equilibrium are of particular interest.

To that end, the rigorous definition of catastrophes in terms of energy instability supplies a generally applicable 'formal' analogy. Structural failures of strength are quantified by external demands exceeding the supply of material resistance (e.g., $Q > R$ in Eq. 1 – a). In contrast, instabilities are inherent in a structure's formal qualities. The described pattern is formally analogous to the 'snap-through' instability of Von Mises trusses and flat arches. Bažant and Cedolin [14] state: "The question of stability may be most effectively answered on the basis of the energy criterion of stability, which follows from the dynamic definition if the system is conservative." The authors present catastrophe theory as a "strictly qualitative viewpoint" analyzing the stability of conservative systems by energy methods as follows: "[Catastrophe theory] seeks to identify properties that are common to various catastrophes known in the fields of structural mechanics, astrophysics, atomic lattice theory, hydrodynamics, phase transitions, biological reactions, psychology of aggression, spacecraft control, population dynamics, prey-predator ecology, neural activity of brain, economics, etc. Simply, the theory deals with the basic mathematical aspects common to all these problems."

Both [13, 14] refer to René Thom's [15] demonstration that in a conservative system with one control parameter only one type of catastrophe is possible (the limit point or snap-through), with two independent control parameters, the fold and the cusp types of catastrophes are possible (asymmetric and symmetric bifurcation). For systems with three control parameters, five types of catastrophes become possible; and systems with up to four control parameters allow at most seven types of catastrophes. The 7 types of catastrophes are called 'elementary'.

Bažant and Cedolin [14] illustrate the snap-though of the von Mises truss as shown in Fig. 4. It is assumed that the bars will not buckle individually under the increasing load P. Rather, a 'global' instability occurs when the potential energy of the elastically deformed system reaches a bifurcation point. Equating to 0 the expression for the second derivative



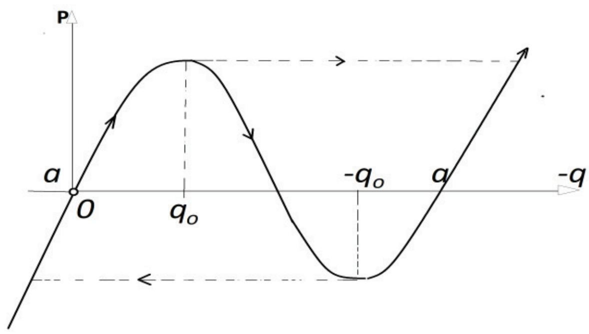


Figure 4 (a) Von Mises truss *(b) equilibrium path*

of the system's potential energy obtains Eq. 2. The truss is stable for $-q_0 \leq q \leq q_0$.

$$\cos q_0 = (\cos a)^{1/3} \tag{2}$$

If energy and money were viewed as the two active parameters controlling the bridge network, each could cause its own type of instability. For four control parameters, for example if intelligence and information were regarded as additional parameters (e.g., representing political restraints), the possible types of catastrophes would increase to seven (Table 4.7.1., p. 300 [14]). Adding further indeterminacy, the inevitable 'passive' system imperfections strongly influence near-instability behavior.

The stability analogy reminds that, apart from failures quantifiable by demand exceeding the supply of strength, infrastructure assets and networks are vulnerable to those of qualitative form. It also cautions that the possible modes of system failure increase (more than linearly) with the number of 'control factors'. The analogy to mechanical instability advances the argument for anticipation and prevention, as opposed to relying on 'emergency response' at 'limit-points' or 'bifurcations'. However, the potential energy of conservative systems depends on measurable and calculable demands and supplies of applied and resisting energy. Hence, an infrastructure network, with its broadly estimated multi-parameter dynamic equilibrium of vaguely quantified and qualified supply and demand, cannot qualify as conservative. Once the stability analogy is not an exact predictor, it can be dismissed as one more doomsday warning. Aspiring Cassandras bear that curse since antiquity.

5 Robust, resilient and sustainable performance

In the terminology of stability theory, the consequences of extreme events escalating to 'national disasters' should qualify as catastrophes. The potentially catastrophic 'flag' history of Fig. 3 was contained both financially and mechanically without reaching disaster magnitude, but it gained 'emergency' status and absorbed substantial local and federal funding that could have served other purposes. Energy and money are the obvious control parameters traditionally quantifying infrastructure network performance. The stability analogy draws attention to the qualitative aspects of that performance. It raises the following questions: What critical parameters best reflect the potential instabilities of the infrastructure products and process, and what variables control them? The response demands terminology integrating the qualitative and quantitative features of energy and money.

According to Henri Léon Lebesgue: "The definition of a new category requires the introduction of at least one new term." As a noun, 'sustainability' remains an abstract quality inviting well-intentioned attitudes and multiple descriptions, but no definition. As defined in Eq. 3 it could quantify the 'performance' of a bridge or a network. As adjective, it is restrained by the parameters of the qualitative politics, economics, environmental protection, and so on, all of which are ultimately monetized in quantifiable budgets. The *sustainability factor* proposed in Eq. 3 essentially measures the long-term affordability of an infrastructure network under the governing social restraints and natural constraints as follows:

$$\textit{Sustainability factor} = \Sigma \textit{ benefits} / \Sigma \textit{ costs} \tag{3}$$

A sustainability factor has significance only relative to the estimated performances of other strategic alternatives (e.g., 'optimal', 'desired', 'prioritized', 'expected'), assessed under the same standards and conditions. The following properties distinguish the sustainability of a performance according to Eq. 3:

– Sustainable performance is not merely a reciprocal cost / benefit ratio. It implies, but is not limited to 'cost-effectiveness' and 'affordability' because neither 'benefits' nor costs are limited to direct immediate activities and services. For example, environmental considerations, still struggling for recognition, can influence both the benefits to the users and the operating costs in ways easier to qualify than quantify over diverging time-horizons. Beyond the known operating (a.k.a. 'direct') expenditures incurred by the responsible owner, the sustained costs include the 'user costs', perceived damages, the consumption and depletion of natural resources.

– Sustainable performance is both qualifiable and quantifiable because the network's optimal, prioritized, and otherwise restrained and constrained 'control parameters' are both calculated and negotiated in the political, economic, engineering and public domains. Hence, sustainability qualifies engineered performance, already quantified in energy and money. As a consequence, alternative solutions are evaluated for environmental, economic and political sustainability assuming (prematurely) that they are similarly feasible in the engineering domain.

– Sustainable performance pertains to *network lifecycles*, rather than to annual budgets and individual projects. As the infrastructure's lifecycle by far exceeds annual budget considerations, sustainability must be perpetually reassessed, optimized (or prioritized) and updated over specified periods. Bridges are sustainable within the transportation network which they re-define. The networks are sustainable within the regional economy. The integration is organic.

By integrating the engineering, economic, political and public aspects of infrastructure performance, sustainability can reconcile the seemingly incongruent constraints and restraints of their domains. Conversely, the absence of such reconciliation can be shown as unsustainable. Tracing sustainability considerations in the UK since 1999, [16] concludes that "a bridge manager's decision-making process will be much more complex when account has to be taken of sustainability. … A procedure for assessing lifetime sustainability is needed to help the manager make consistent and good decisions."

Consistently with Lebesgue's postulate, the terms robustness and resilience have qualified, and to a degree quantified, 'performance-based' structural design within the energy and time constraints of engineering specifications prior to introducing 'sustainability'. Robustness is defined as the ability of a structure or network with an impaired resistance to redistribute its supply to meet the load demands of 'extreme events' in constrained time. In the explicit forms of redundancy and ductility, robustness redistributes and sustains the load demands in the time and space of defined assets and events. Bruneau & Reinhorn [17] define resilience as "the ability to prepare and plan for, absorb, recover from, and more successfully adapt to adverse events". Hence, resilience describes the capacity of the network to deliver services over extended lifecycles.

Networks consist of assets and assets are networks of elements. Hence, sustainability implies robustness and resilience on both the individual and group levels in both the mechanical and financial domains. The terms are not ratable according to any qualifying or quantifying scale so far. FHWA [18] has advanced bridge management towards the standardizing and codifying of their assessments. Figure 5, proposed in Yanev [19] illustrates the following realistic lifecycle of the engineered asset(s) in the plane of energy / robustness and time / resilience, under 'normal' demands and an extreme event.

Current design specifications prescribe bridge performance in terms of strength, stability, ductility, redundancy, and criticality. A performance may deteriorate over time at a variable rate, depending on many external and intrinsic factors. Ensuing disruptive 'extreme events' can be external natural disasters, or internal structural non-performance. Robustness is the structural capacity to survive the energy onslaught of extreme events at discrete times with residual functionality. Resilience pertains to the process of network response, not only in terms of energy, but also in terms of money and extra-monetary considerations over extended, but nonetheless foreseeable periods. At both project and network levels, robustness and resilience imply redistributing a constrained supply of resistance in response to an expanded demand, as do structural redundancy and ductility. The structural condition ratings illustrated in Fig. 2 imply a moderate decline of robustness on the project level, but the sufficiency ratings indicate deficient serviceability and hence, a waning sustainability. Under the incongruent engineering constraints, economic and political restraints, post-event recovery may reach a sustainable level of structural robustness and network resilience or merely restore a state preceding the next crisis.

Assuming, as in Eq. 1, that engineering and economics are constrained and restrained in the 2-D space defined by the 'control parameters' of energy and money, sustainability adds the 'third dimension' of time-space necessary for evaluating and managing infrastructure network performance. Figure 6 illustrates the 3-D space in which engineering, economics and politics can jointly manage a sustainable infrastructure. Rotating the axes of energy, money and time-space in the respective planes of engineering / economics, economics / politics, and engineering / politics obtains the new axes of robustness, resilience and sustainability. By integrating energy, time-space and money, the new 'control parameters' of the social and physical performance restrain engineering, economics and politics into collaborating. As 'control parameters' intelligence / information would contribute further complexity beyond the present scope. The separate sets of intelligence / information inherent in the Energy, $ and Time-Space dimensions adopted by engineering, economics and politics can account for their occasional contradictions.

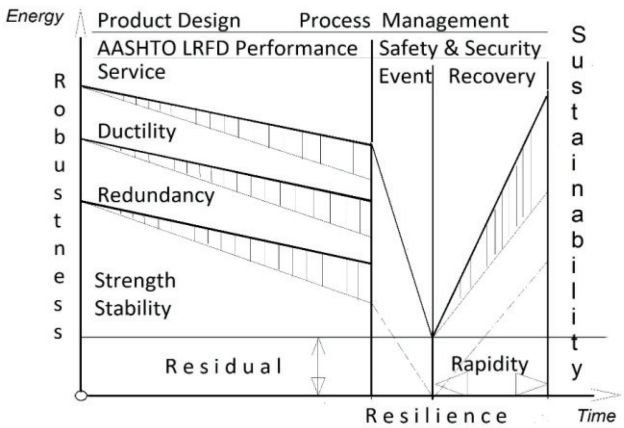


Figure 5. Robustness, resilience and sustainability of bridge products and network performance under routine and extreme service demands

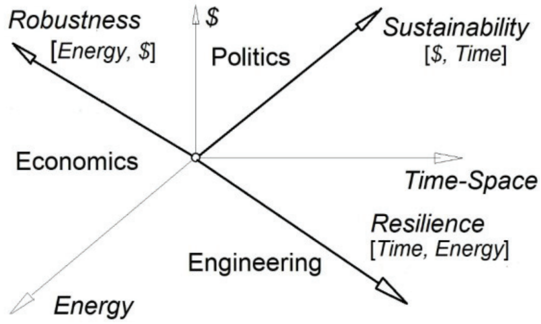


Figure 6. Control parameters of the infrastructure performance space: robustness, resilience and sustainability in engineering, economics and politics

6 Engineering and economic prioritization of maintenance and reconstruction

Bridge management operating options can be reduced most generally to maintenance and (re)construction. Under the disparate political, economic, engineering, and other restraints and constraints governing the process these options cannot be rigorously optimized. Most major failures are caused by more than one critical deficiency. Bridge-related hazards proliferate catastrophically due to deferred maintenance and delayed reconstruction, both of which are precipitated by shortages of money (and occasionally, information). Once arising however, they command funding allocation. Economic decisions take for granted and occasionally disregard the analysis of the engineering information during periods perceived as stable but rely on it incontestably in catastrophic extreme events. Between 1991 and 1998 the reported up to 3,200 annual flags could not have been addressed physically ground-up if top-down analysis had not reviewed and prioritized their urgency. However, the decline in structural condition ratings prior to 1987 and the corresponding mild increases in potential hazards were signaling the approaching instability. The following lifecycle model of bridge network supply of performance and demand for maintenance and rehabilitation illustrates the point.

If the condition of a bridge network R with total deck area A were in a 'steady state' from one year to the next, with ratings distributed close to uniformly along the scale, Eq. 4 should describe the equilibrium between their deterioration rate r and the quantity A_{rec} entering reconstruction annually. The ratio A / A_{rec} expresses the benefit / cost sustained by both the community in terms of service reduction and by the responsible owner in terms of construction costs. The improvements due to repairs without closures, discussed in Example 18 [9], are ignored herein.

$$A / A_{rec} = \Delta R_{rec} / r + n \tag{4}$$

where:

- A is the deck area of the bridge network
- A_{rec} - deck area entering reconstruction annually
- R - average bridge condition rating on the NYS rating scale (7 – 1)
- ΔR_{rec} - average total change of R of A_{rec}
- r - annual rate of bridge deterioration ($\partial R/\partial t$)
- n - average duration of reconstruction in years

Inspection records of the period suggested the following values:

$A \approx 1{,}500{,}000\ m^2$; $r \approx 0.2$ *points*; $\Delta R_{rec} \approx 4.5$ *points*; $n \approx 3$ *years*

Substituting the preceding values in Eq. 4 obtains a condition rating equilibrium requiring $A_{rec} \approx 0.04A = 58{,}824$ m^2. Reconstructing annually $n\ (0.04A) = 0.12A \approx 170{,}000$ m^2 is physically and economically unsustainable, hence tantamount to a network failure. Financially, at an average reconstruction cost of 10,000 \$US/m^2, the demand for annual expenditures would amount to approximately \$US 600 million (1989) and the benefits would take n years to materialize. Inevitably, hazards must be mitigated as they arise. After the fatal accident at FDR on June 1, 1990, \$US 50 million were dedicated to addressing similarly rated bridge deck conditions, affecting half of the city bridges. Over 20 years, the four East River crossings were rehabilitated with partial traffic closures, each absorbing more than \$US 1 billion. By the year 2000 capital reconstructions reached the annual cost of \$US 600 million, effectively reducing annual flag numbers to the manageable 1200. In 2020 the annual budget of the agency approached \$US 1 billion, predominantly in reconstruction costs.

In Fig. 7 the potential hazard history of Fig. 3 is reduced to the polygon A – B – C – D – E. Since hazards and their mitigation expand the operating costs quantifiably while reducing the service both quantitatively and qualitatively, inverting their history can be regarded as reflective of the network's sustainability. Given sustainability's qualitative nature, it could be scaled to the anti-symmetric pattern illustrated by the dotted line of the polygon A′ – B′ – C′ – D′ – E′. Its path reflects the drop in the supply of services. Inverting the graph of Fig. 5 would produce a similar pattern.

The five stages described by the polygon A – B – C – D - E in Fig. 7, as well as the flag history in Fig. 3 can be considered as symptomatic of a catastrophic event. They include an apparent equilibrium of services and expenditures (A – B), expanding demands for corrective actions (B – C), a state of maximum demand (C – D), a decline in the demand (D – E), followed by a new equilibrium at a higher supply of services / demand for expenditures. A formal similarity is discernible between the potential energy of 'conservative' mechanical systems and the 'sustainability' of the bridge network described by the inverse polygon A′ – B′ – C′ – D′ – E′ in Fig. 7.

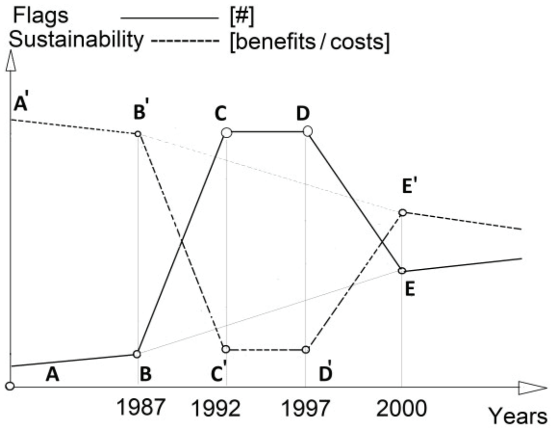


Figure 7. Potential hazards and respective sustainability of a bridge network (not to scale)

Since neither the quantifiable energy and money, nor the qualifiable intelligence and information of the network are conservative, infrastructure managers, their critics, and the media assess its condition in broadly varying terms, ranging from 'challenging' to 'catastrophic', with comparable conviction. Modifying the standards of the descriptive bridge condition and prescriptive potential hazard assessments in response to an infrastructure crisis would be a non-conservative equivalent of the fiscal manipulations attempting to stave off a financial collapse. Particularly salient are the following features of the Von Mises truss analogy:

− The potential energy of the conservative system equals the difference between the energy of the loads and that of the elastic structural deformation. By analogy, both increasing demand and reduced supply can cause instability. The established supply of and demand for network services and needs could be assumed as constant over relatively short periods, however in general, management should anticipate their growth.

− Under increasing load, single degree of freedom (SDOF) systems are stable while the slope of the potential energy is positive and unstable while it is negative. A declining system robustness (analogous to 'stiffness') is potentially unstable. The escalating demands related to potential hazards trigger the 'extreme event' of A′ – B′ – C′ – D′ – E′ in Fig. 7, however if they were avoided, the slope B′ – E′ still remains negative and hence, tends towards unsustainable.

− In a 'snap through' instability the two bars of the Von Mises truss do not buckle individually. The overall geometry of the 2-bar system becomes unstable due to 'unsustainable' elastic deformation. Thus, network instability can be due to the failing robustness of one critical link or to the decline in the overall resilience of the network (as in unsustainable traffic volume and structural safety demands).

The formal analogy between network sustainability and structural stability draws attention to the period denoted as A – B in Figs. 3 and 7, corresponding to 1982 – 1987 in Fig. 3, as well as the period of steady decline preceding the 'extreme event' in Fig. 5. In terms of 'snap-through' instability, this is the period when potential energy is approaching instability due to 'elastic' deformation of the system. The bridge condition and sufficiency ratings of Fig. 2, and the declining robustness of Fig. 5 suggest that the resilience is approaching critical un-sustainability. Certain languages use the same word for 'stability' and 'resilience'. Beyond point B in Fig. 7 the system is already in a catastrophic 'extreme event' when only emergency measures are appropriate. Also evident is that beyond a certain loss of robustness (Fig. 5) and sustainability (Fig. 7), full replacement becomes the only option.

Possible alternatives of operating costs for the considered period are plotted not to scale in Fig. 8. The corresponding numerical values in Table 3 are tentative and non-homogeneous because reconstruction is funded by federal, state and local sources, whereas maintenance was funded only locally at the time. The numbers include inflation. Nevertheless, they realistically quantify the monetary implications of the alternative strategies balancing reconstruction / maintenance, as well as the increase of total expenditures from initial to ultimate. The New York City Bureau of Bridges was founded in 1988 to a large extent in response to the looming bridge crisis. By then the 'extreme event' was in progress and the paths A – B (and A' – B') of Fig. 7 were physically unsustainable. Mitigating the hazards to the public was the emergency priority. By the year 1997 however, both options, denoted as 1 and 2 in Fig. 8, were viable. By 2000 the difference is distinct. Reconstruction and maintenance could continue along path 1 at the established ratio. Alternatively, preventive maintenance could be radically increased, reducing the demand for reconstruction over time, as in E' and path 2. The Report [20] recommended dedicating 1% of the network's replacement cost to annual maintenance, amounting to approximately $US 100 million (2000). That maintenance should extend bridge life from 40 to 120 years, implying $r \approx 0.067$. The implied effectiveness of the investment in maintenance is notoriously prone to the 'system imperfections' of poor execution.

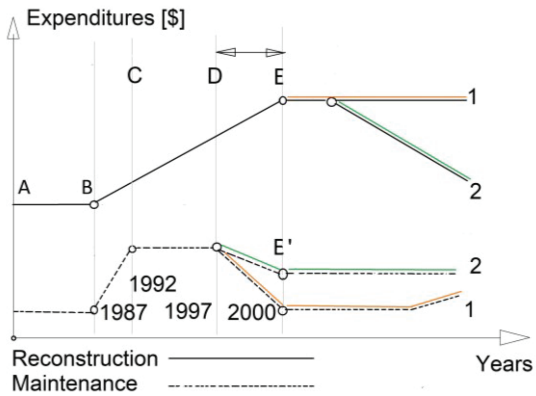


Figure 8. Alternative funding to the reconstruction and maintenance of a bridge network (not to scale)

Table 3. Hypothetical operating costs for the NYC bridge network (1987 – 2000) in million $US

Year	**1987**	**1988**	**1989**	**1990**	**1991**	**1992**	**1993**	**1994**	**1995**	**1996**	**1997**	**1998**	**1999**	**2000**	***Alt.***
Reconstruction	90	100	110	120	130	140	160	180	200	250	300	400	500	600	*600*
Maintenance	10	20	50	50	60	60	60	60	60	60	50	50	50	50	*100*
Total	100	120	160	170	190	200	220	240	260	310	350	450	550	650	*700*

Given the high direct and user costs associated with reconstruction and hazard mitigation, the cumulative maintenance and reconstruction expenditures represented by path 2 offer superior long-term sustainability. For example, let more effective maintenance and reconstruction modify the terms in Eq. 4 as follows:

$r \approx 0.1$ *points*; $\Delta R_{rec} \approx 5$; $n \approx 2$

Then $A_{rec} / A \approx 0.02$. Hence, the immediate investment in more effective maintenance reduces future reconstruction costs and improves community benefits two-fold. Improved long-term sustainability is popular during post-event recovery periods, however the demands of the same recovery preempt immediate investments in it. Moreover, costlier maintenance adds to the budget without reducing imminent reconstruction needs. Such is the $US 50 million increase in the Alternative budgeting for the year 2000 (last column of Table 3). The customary administrative preference for reconstruction contracts over in-house maintenance similarly influences management choices. By 2008 NYC DOT raised bridge conditions above the NYS DOT [4] rating of 3 (not functioning as designed) primarily by intensifying reconstruction and struggles to maintain that dynamic equilibrium since.

7 Discussion

Using the ‘stick-slip’ terminology of mechanics, Umberto Eco [21] advises that “History is sticky and slippery. We must always keep in mind that tomorrow’s catastrophes are secretly ripening today.” In the present view, in order to be understood and managed, crises must be viewed and dimensioned integrally as current processes and products of past ones. Historically, infrastructure network management has advanced mostly after the ‘disasters waiting to happen’ happen. Possibly explicable in the cases of the relatively random natural disasters, that course appears irrational in ‘extreme events’ extended over time, as in the case of the predictable structural decline. Why infrastructure management fails to adopt stable strategies of prevention until the instabilities illustrated in Figs. 3 and 6 become unsustainable? An explanation is sought in the insufficiently scrutinized incongruence between the constraints and restraints of energy and money governing supply / demand in engineering and economics, illustrated in Fig. 1 and Eq. 1. If engineering, economics, politics, and popular sentiment fail to reconcile their different attitudes towards the common ‘control parameters’ of energy, money, and time, they shall continue to court catastrophes.

In an infrastructure network, a perfect balance of the physically constrained engineering quantities and socially restrained economic, political and other qualities is not only impossible, but unsustainable and unstable. Conservative mechanical structures can suffer from catastrophic instability unless their initial and deformed shapes are analyzed with respect to the potential energy of the acting loads. In a formal analogy, the services and performance of a transportation network, traditionally quantified in terms of money and energy can be also qualified in terms of a dynamic sustainability, expressed as a function of its robustness and resilience in space and time.

Bažant and Cedolin [14] caution: “The study of structural stability is often confusing because the definition of structural stability itself is unstable. … one definition of stability – the dynamic definition – is fundamental and applicable to all structural stability problems. Dynamic stability analysis is essential for structures subjected to nonconservative loads, such as wind or pulsating forces. Structures loaded in this manner may falsely appear to be stable according to static analysis while in reality they fail through vibrations of ever-increasing amplitude or some other accelerated motion.” Bažant and Cedolin [14] reiterate that in inelastic systems instability can occur below the critical loads but may follow stable paths. Since a transportation network is neither ‘conservative’ nor ‘elastic’, the stability analogy is purely formal but usefully underscores the following critical imperatives of infrastructure management:

- The assets must be managed as a process in time, as well as a network of products in space.
- A sustainable process (as a stable structure) will depend at minimum on product robustness and process resilience, which in turn can be quantified in the traditional control parameters of energy and money. Implicit but critical, intelligence and information add significantly to the modes of instability.
- Given the ‘energy’ and ‘money’ dissipation characterizing the supply and demand governing an infrastructure network, a ‘horizontal’ slope of the modeled parameter a fortiori corresponds to a potentially unstable equilibrium.
- At losing stability, the Von Mises truss ‘snaps through’ from one stable state to a geometrically opposite one. It is assumed that its bars will neither buckle in compression nor rupture in tension. In an infrastructure network such dynamic transitions could be compared to rapidly escalating demands for money and energy, possibly exceeding the economic and productive capacity of the system and causing local failures.

8 Conclusions and directions

The pursuit of sustainability advances by defining it. A World Summit on Sustainable Development was held in Johannesburg in 2002, following a related event in 1992 at Rio de Janeiro. In 2010 the US Report [22] focused on sustainable development of chemicals, transport, mining, waste management, and sustainable consumption and production. In 2011 the Office of Sustainable Development at the United Nations (UNSOD) established 17 goals (SDGs), emphasizing least developed countries (LDCs). The subject can advance from well-intentioned general directives to specific tasks if ‘sustainability’ and its constitutive ‘robustness’ and ‘resilience’ are defined in consistent and accepted qualified and quantified terms. To that purpose, the present view reduces the scope to the management of a local transportation infrastructure of a metropolis. Even on that scale, an equilibrium of supply and demand in the disparate terms of energy and money cannot be rigorously established. A general analogy between mechanical instabilities and crises in other socially critical domains however can be discerned and qualified in terms of sustainability, robustness and resilience.

Over the considered period the robustness and resilience of a bridge network slid from stable through neutral equilibrium to potential instability, whereas the established equilibrium suggested no potential instability until public safety demanded emergency funding. Below a certain qualitative level, declining bridge condition ratings trigger an increase in potentially hazardous conditions and hence, an economic ‘bifurcation’ quantifiable in money. Direct and user expenditures sustained a loss of quality of life, until a relatively manageable stable state was reached at higher annual direct costs.

The reported flag forecast, as well as most current models of bridge condition deterioration nationwide, were and remain based on the 10 and the 7 – level qualitative condition ratings of [1] and [4]. The recent transition to the 4 quantified element condition states of [3], all of which can coexist in the same element (of a span or the bridge), introduces a critical discontinuity in the invaluable NBI database. Changes in the NYS DOT flagging procedure have had a similar effect. According to [5] non-structural conditions are no longer 'flagged' and utilities are treated separately. The forecasting reported herein would have been impossible without the preceding decades of consistent qualitative assessments. Duplication and re-distribution of effort have ensured most engineering successes, whereas their elimination (advertised as streamlining by fiscal-oriented management) has caused many failures. A single perfect condition assessment system does not exist. Management, as all other engineering branches, becomes robust, resilient and ultimately, sustainable, by relying, as much as possible, on redundant and complementary strengths.

Given the heterogeneous, inherently discontinuous information, a rigorous, universally applicable algorithm could not have been developed then nor is available currently. In the words of Von Neumann and Morgenstern [11]: "Even in sciences which are far more advanced than economics, like physics, there is no universal system available at present." The authors highly recommend quantification but acknowledge its limitations. Engineering management must maximize reliance on science, but, particularly under severe constraints, has to produce art, as the French term *ouvrages d'art* implies. As extensively quantifiable as decision support might be, managers contribute, if at all, by executing qualitative decisions. No generic algorithm could have supported a legitimate budget request. There is no substitute for qualitative managerial expertise and soundly motivated choice. The element level condition states adopted on the federal and state levels in [3, 5] supply a valuable quantifier but not a substitute for the qualitative assessments. The complex information and far-reaching implications contained in the flag history of Fig. 3 demonstrate that no single system of parameters can fully capture the diverse and incongruent supply and demand inequalities governing the engineering and economic management. Isolated violations of the constraints in Eq. 1 may not be critical, whereas approaching their breach on a network scale would guarantee a crisis. Therefore, it becomes imperative to examine the available engineering and economic indicators in order to discern a potential crisis while it can still be averted, and if necessary, identify new such indicators and 'control parameters'.

Since the reported period FHWA has acknowledged recognition by several innovations. Bridge life-cycle performance has become a central design consideration. Rehabilitation, repair and maintenance activities were integrated in Bridge Preservation [6], eligible for federal funding on par with capital reconstruction. (The repainting of a major bridge can cost hundreds of millions of $US and hence, qualifies as a capital project.) As [18] suggests, condition assessments remain a work in progress. Since instabilities in non-conservative systems are even harder to identify, they should be precluded by broad margins.

The robustness and resilience of transportation networks are gradually gaining forms allowing for their qualitative & quantitative assessments. By integrating engineering, economic, and environmental criteria, they lend a manageable meaning to the 'third dimension' of sustainability, beyond elementary cost-effectiveness. Sustainability, quantified and qualified in engineering and economic terms, becomes indispensable for managing infrastructure performance. Sustainable lifecycle strategies anticipate and prevent relapses to the potentially unstable conditions of NYC bridges in1987. Unmanageable project-level losses of robustness and resilience become unsustainable and hence irreversible on the network level. System 'imperfections' near the points of instability invalidate routine expectations. Consequently, infrastructure planning requires at a minimum a 20-year horizon in order to anticipate and avoid the 'poli-crises' currently discussed at international gatherings of economic experts.

Figures 3 and 7 imply that post-extreme event equilibrium is attained at higher costs and hence, diminished sustainability. This phenomenon corresponds to the endemic inflation, the ubiquitous entropy, and the traditional lament for the 'good old days' when 'things were better'. The network sustainability improves by expanding benefits and reducing expenditures, again reducing to superior robustness and resilience of engineering products and economic process.

In 2021 $US 1.3 trillion were allocated by Act of Congress to rebuilding the national 'hard' infrastructure towards an unquantifiable but presumably sustainable level. Emphasized are the mythical "shovel-ready" projects, which President Barak Obama has called "nonexistent". There is no explicit mention of the ensuing perpetual maintenance costs, however every new construction must imply a financially sustainable commitment to maintain the product and the process in robustly and resiliently performant condition over the designed useful life. Since the transportation infrastructure is part of the general social fabric, along with many other domains, such as the energy, chemical, natural resources, waste disposal, and social services, optimization invariably yields to prioritization, and can easily degenerate into emergency management by triage. The deadlocked negotiations over spending and national debt limits between the Legislative and Executive Branches of U. S. Government in 2023 demonstrate the political precarity that engineering and economics management must be able to neutralize. A qualitative and quantitative reconciliation of the energy and money supply and demand between these two essential infrastructure management domains would ensure the resilient, robust and sustainable management of the 'hard' infrastructure that society depends on.

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Preliminary report

The influence of the 2020 crisis on the demand for traditional and novel construction and building materials in Serbia

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ABSTRACT

Even though the specific COVID-19 consequences for sales have been extensively discussed, no academic research has been done on how the pandemic has affected consumer choice and purchases of construction and building products by private individuals. This research was conducted to fill in the gaps in the body of knowledge and advance understanding of how the crisis has impacted wages, market prices, and material usage in the construction industry in a developing country. The data are collected through the use of a questionnaire survey. The respondents shared their experiences between the period before 2020 and after the beginning of the crisis up until the end of 2022 and showed that purchases of these products decreased during the lockdown and afterward. The obtained results were analyzed using statistical tools, namely frequencies, descriptive statistics, and constructs. This study reveals a high interest in using novel materials but also a desire to be more informed on the details and their potential benefits. The results present a first-of-a-kind approach that will help further development in this branch of the industry by following the needs of potential private customers in a developing country. Further studies would need to include not only contextual but also personal factors that influence environmentally friendly choices.

1 Introduction

The construction industry represents a great threat to the natural environment and thus is under a lot of pressure to become more sustainable, considering the high consumption of energy and raw materials and its considerable contribution to global greenhouse gas emissions [1,2]. Besides, this sector generates huge quantities of waste after the construction and demolition phases [3,4] and requires a high-profile change.Nearly 30-40% of total solid waste in the world is from construction and demolition processes, whereas its production only in Europe is around 0.175 billion tons/year [3]. Developing countries are estimated to produce more than 10 times the quantities produced in Europe [3]. Sustainable solutions are increasingly available; however, they do not appear to be generally used.

The UN Sustainable Development Goals, which call for consideration of environmental, social, and economic life cycle sustainability in buildings, are closely tied to the need for sustainability adjustments in the residential construction sector [5]. Seen in this light, the possibility of using different waste or lower-quality materials in production has been examined for decades [3, 4, 6, 7], increasingly implementing mathematical modeling of large datasets [8]. Various new production methods are also being tested, such as geopolymerization [9, 10]. Life cycle assessments are recently being intensively performed, to judge the impact of a certain product on the quality of the living environment [11-13]. In addition, the introduction of the ecological label on certain products from this branch of industry has become mandatory [14] and is a good practice to bring relevant information to the customers [15].

In 2019, Serbia's economy was in a mediocre state, with real GDP growth of 3.2% and the lowest 10-year unemployment rate of 10.5% [15]. Early in 2020, the nation's finances were in much better shape thanks to considerable reductions in its fiscal deficit and external debt [16]. A global pandemic that had never been seen before began to spread in 2019 [17]. The effects of COVID-19 in Serbia had an increasingly negative impact on workers in the informal economy and smaller enterprises. With the Russian-Ukrainian war beginning on February 24, 2022, the world economy has continued to undergo major changes [18]. The cost of building materials in Serbia has dramatically risen since 2020 [18-20]. A further increase is anticipated given the rising cost of energy [20] and the fact that it contributes a high amount to the price of building materials, especially steel and concrete [21]. The result was an increase in the price of

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residential space per square meter, which is estimated to be 1.8 % for new dwellings when 2019 and 2020 are compared [22]. Top-down and bottom-up economic shocks occurred, and substantial changes in corporate and personal circumstances affected both domestic and global supply and demand trends for products and services [17, 18].

The previously published surveys in the field of construction and building materials and products are scarce, while many include experts in the field and are based on construction projects in developing countries [18,23]. Only one partly similar 100-respondent survey was conducted in Russia in 2016, aiming to find the behavior patterns of private consumers of construction materials. The main conclusions were that price and quality were the key criteria and that a low share of respondents were oriented toward environmentally friendly materials [24]. Other studies were concerned about specific environmentally friendly solutions (using construction and building materials incorporated with dredged sediments or construction and demolition waste) by consumers in Belgium [15] or contractors in China [25]. While consumers were mainly worried about the quality and chemical resistance of such products [15], contractors were mainly driven by government measures. Furthermore, a review study on willingness to use construction and demolition waste containing materials determined “negative attitude” as the main personal boundary, and as a contextual problem, price and quality were found [26]. Furthermore, the factors influencing the willingness to use recycled building materials based on the perceptions of the main stakeholders in the construction industry are studied in New Zealand. Results indicated that price and self-satisfaction are of the highest influence, while the choice was also found to depend on the age, gender, and income of the respondents [27]. In addition, a study dealing with general green product consumption in India was based on an in-depth questionnaire survey of 20 professionals. The price and quality of the products were determined to be a major concern for customers of green products [28]. Another similar study was done in Germany with 306 participants and aimed to find out the decision-making process of individuals to buy environmentally friendly construction products [29]. Their findings show that although customers are generally interested in sustainable building goods, they do not have a comprehensive understanding of the term.

This study focused on a randomly chosen private group of people to observe their experiences and opinions on the subject. A minimal sample size required was calculated from the formula [30], and it was determined that a sufficient number of 391 respondents to describe a 7 million-person nation answered the questionnaire [18]. The following questions are addressed: In what ways has the crisis affected salaries, market pricing, and material consumption in the building sector? Which factors influence the choice of construction and building materials in Serbia? What is the connection between being interested in ecological and novel products and purchasing them? A special emphasis in this work is given to novel and ecological materials and further directions of sustainable development in this industry.

2 Methodology

The inquiry was open to all Serbian citizens who were at least 18 years old. The goal was to compile a representative sample of people from a range of demographics, including age groups, specialties, and levels of ownership of flats, houses, and cottages. An objective and wide picture of the situation in Serbia is expected to be seen since the survey was not based on respondents like experts in this field. Industry and business specialists were given a draft of the questionnaire for discussion and improvement. After the agreed-upon revisions had been incorporated, a small random sample of participants completed the survey to ensure clarity and improve the study's validity. The list of questions is given in the Appendices. The answers were gathered using multiple methods, such as an online and paper-form questionnaire, between January and September 2022. Only those respondents who fully answered the survey were included in the analysis. A total of 391 respondents were deemed qualified to describe a country like Serbia with a population of 7 million people [18].

The first group of questions was primarily concerned with important sociodemographic information (age, gender, education, occupation, and salary satisfaction). Additionally, the respondents fulfilled the information on possessing a residential or guest property and what kind of home they resided in (an apartment or house, rented or owned). Another set of queries focused on the purchase of building materials and products before and during pandemics and major world crises. The goods in question were divided between construction products with a specific shape (bricks, tiles, sanitary ware, carpentry, etc.) and building materials (cement, glue, paint, etc.). During this session, a quantitative seven-point Likert scale survey was given out as needed. The final set of questions aimed to find out if respondents had ever used novel construction materials and how likely it was that they would do so in the future.

Statistical analysis is employed to explain and study the collected data using the IBM SPSS 22 program. Exploratory data analysis, such as frequencies and descriptive statistics was used to analyze the obtained database. Furthermore, to determine the number of components (constructs) that dominate the observed variables and, consequently, options for data aggregation, a principal factor analysis was carried out [15, 31].

3 Results and Discussion

The initial round of questions concentrated on crucial sociodemographic information, including age, gender, education, profession, income satisfaction, and changes in earnings following the crisis starting in 2020 (Q1-Q14, Appendices). The detailed results of these extended socio-demographic results for a tested group are presented in a previous study [18]. The age group of the respondents with the highest percentage (30.2%) was 31–40, while the least numerous group (0.3%) was aged between 71–80. Those with a college degree had the fewest percentages (1.0 %). In addition to office professionals (11.5%), doctors and medical personnel (14.3%), and engineers (16.9%), the group also included professors/lecturers (10.7%) and scientists (8.4%).There were a reasonable number of people in various professions and also those who were unemployed, which enriched the database's diversity. Women with university degrees and residents of the city with a population of more than 2 million (Belgrade, Serbia) made up the majority of the participants [18]. Respondents in questionnaire research carried out in Croatia had a similar sociodemographic distribution [32].

A 7-level Likert scale is used to gauge salary satisfaction, which is primarily expressed as average (27.6% of the respondents). Their income generally grew (51.4%) after the start of the pandemic and the current crisis, which might be attributable to advances in the workplace—a factor that was ignored in this study [18]. The respondents were also questioned about the kind of home they occupied and if they owned a residential or a guest house. Most interviewees claimed they had never relocated before. The vast majority of survey participants do not own a private residence, and of those who do, the bulk of the homes were constructed more than 40 years ago [18].

The second set of questions focused on the purchase of building materials before and during pandemics and major world crises. The majority of respondents (32.7 %) experienced that a product's price is averagely correlated with its quality, that the price/quality relationship has not considerably changed since the crisis, and that the cost of construction and building materials has significantly grown [18]. This is consistent with a report by the National Association of Home Builders, which claims that building material costs in the US grew by 20.4% annually and by 33% overall since the pandemic began [33]. The respondents mostly purchased these products 2-5 years before the pandemic (41.4 %). An increased percentage of study participants (54.5 %) stopped buying the products of concern once the pandemic and crisis started. A more thorough picture of the situation, seen from the point of view of private individuals, in the analyzed country, was created.

The third set of inquiries aimed to assess the methodology by which construction and building materials and products were purchased before and during the crisis (Q17-Q21 in the Appendices). Before 2020, 34% of respondents chose these products based on a fair price/quality relationship, which was found to be the same in previous studies [15, 24]. The smallest share of people bought the most expensive (4.6 %) or products from famous firms (9.7 %). This result could be a problem when accepting waste-added products since private purchasers tend to put confidence in those when they trust the manufacturer [15], which might be different from the point of view of contractors as primary purchasers of these products [25]. If particular knowledge is absent, trust is regarded as crucial [15]. During the crisis, these products were mostly not purchased by private individuals, and among those that did, they again chose a fair price/quality relationship (26.1%) [18].

Furthermore, the respondents' usage of novel building materials as well as their likelihood of choosing to do so in the future are asked (Q24-Q28 in the Appendices). Presumably, the respondents (78.5%) do not presently use novel materials, while only 9 % claimed they do [18]. Among those that are aware of using these products in Serbia, the majority of participants use shaped products (58.14%), while the rest (41.86%) use non-shaped materials [18]. The majority of respondents claimed they are mostly interested in using novel and ecological materials at an above-average Likert scale level, while the scope of work required, the price of a product, and their knowledge of the benefits would influence their choice to a similar degree (Fig. 1). In conclusion, it is seen that the producers ought to be more open about this topic. This is consistent with other studies and surveys carried out in the industrial sector [15, 24] where purchasers from developed countries expressed concern about the quality and chemical inertness of the waste-added products.

Another issue is that there are not many of those clearly labelled environmentally friendly goods on the market [26]. A product's price increase would result in more information being needed by prospective customers, which can present a drawback to adopting Eco-labelled products [18, 34]. Performance and return on investment will improve if construction companies and individual customers are aware of the key advantages of using environmentally friendly construction and building products. The demand-supply dynamics in this market segment will improve as the potential benefits become more apparent [35].

The answers considering construction and building materials and products were calculated per response (Fig. 2) since some of the answerers offered multiple responses to

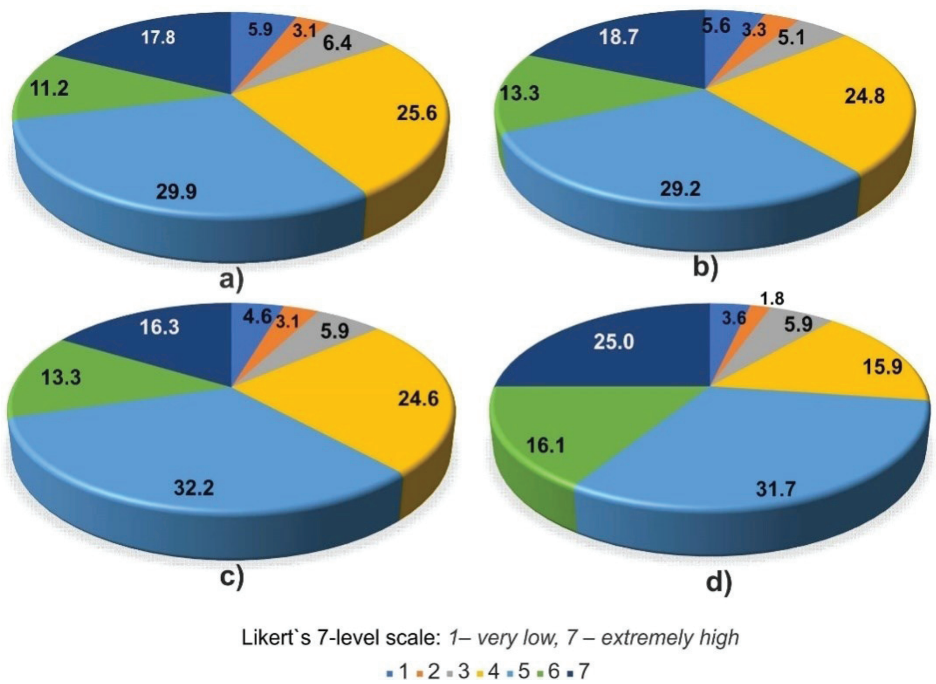

Figure 1. Based on what did the respondents choose construction: a) Materials before the pandemic and world crisis (Q17), b) Products before the pandemic and world crisis (Q18), c) Materials during the pandemic and world crisis (Q20), and d) Products during the pandemic and world crisis (Q21)

Q28. The responses about all novel material usage are summed to 100%. Considering shaped products (Fig. 2a), most of the answers were related to relatively novel carpentry options (13.1%). The next choice in line was kitchen work surfaces like HDMR wooden boards and nano-composite or onyx stone. (8.2 % of all the answers). Masonry and covering building products like siporex concrete blocks and roofing tiles, or eco-separate walls, were a 6.6 % choice. Floor covers, including eco-ceramic tiles and other novel materials, were also utilized. There are also rare examples of using green and fiber-reinforced concrete, structural timber products, and solar panels, accounting for 3.3.% of the responses each (Fig. 2a). The rarest were the recycling of construction aggregates, eco-lightning, and eco-electrical installation. Among non-shaped products (Fig. 2b), most of the answers (11.5%) were related to different kinds of paint (acrylic, specialized, or polymer). The next choices in line were materials used for fungal treatment purposes (9.8%) and coatings used for thermal insulation and protection of wooden materials. Waterproofing agents, novel adhesives for parquet and ceramic tiles, and polymer cement mortar were also utilized. The low practical acceptance of novel products is not surprising considering that having a general awareness of the environment does not guarantee that one will act in an environmentally friendly manner [15]. Furthermore, market demand determines whether resource recovery efforts are successful [26]. However, a personal attitude and being aware that everyone influences the quality of the global environment through their choices is a powerful motivator for those choices [36].

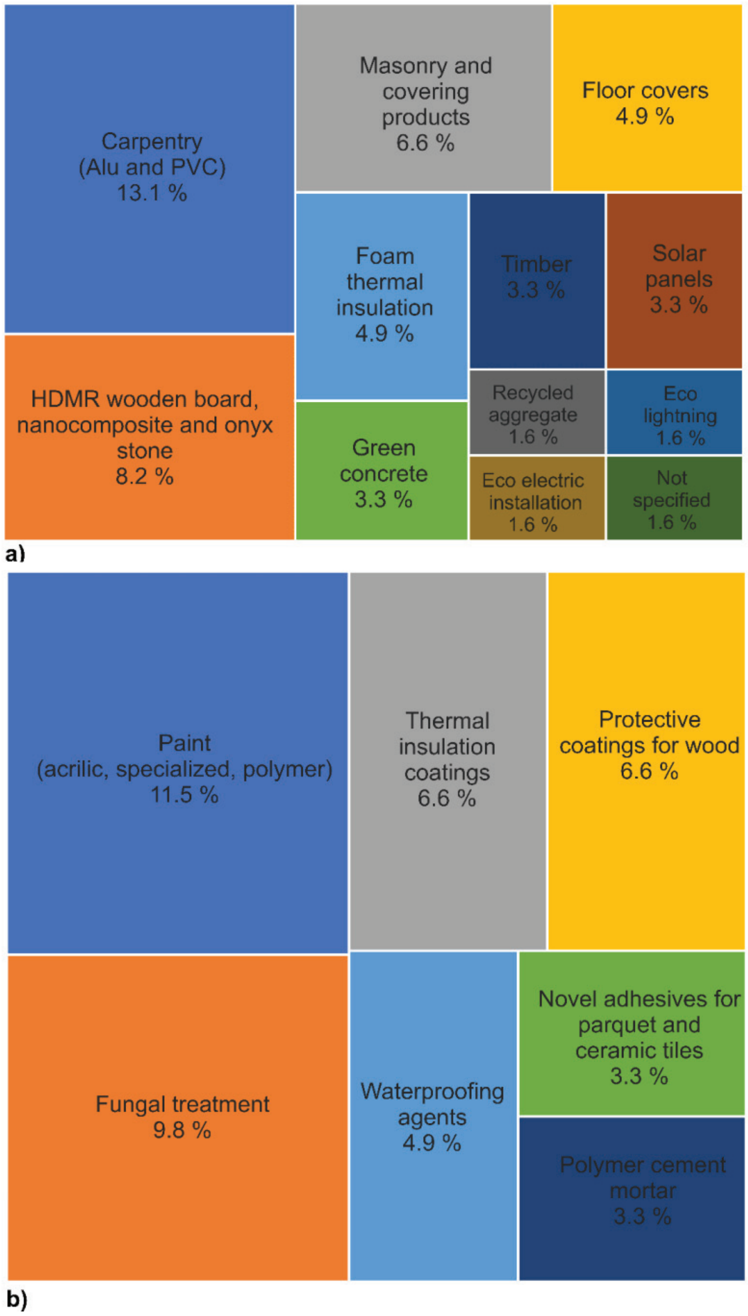

Figure 2. The novel construction and building materials usage: a) Shaped and b) Non-shaped products

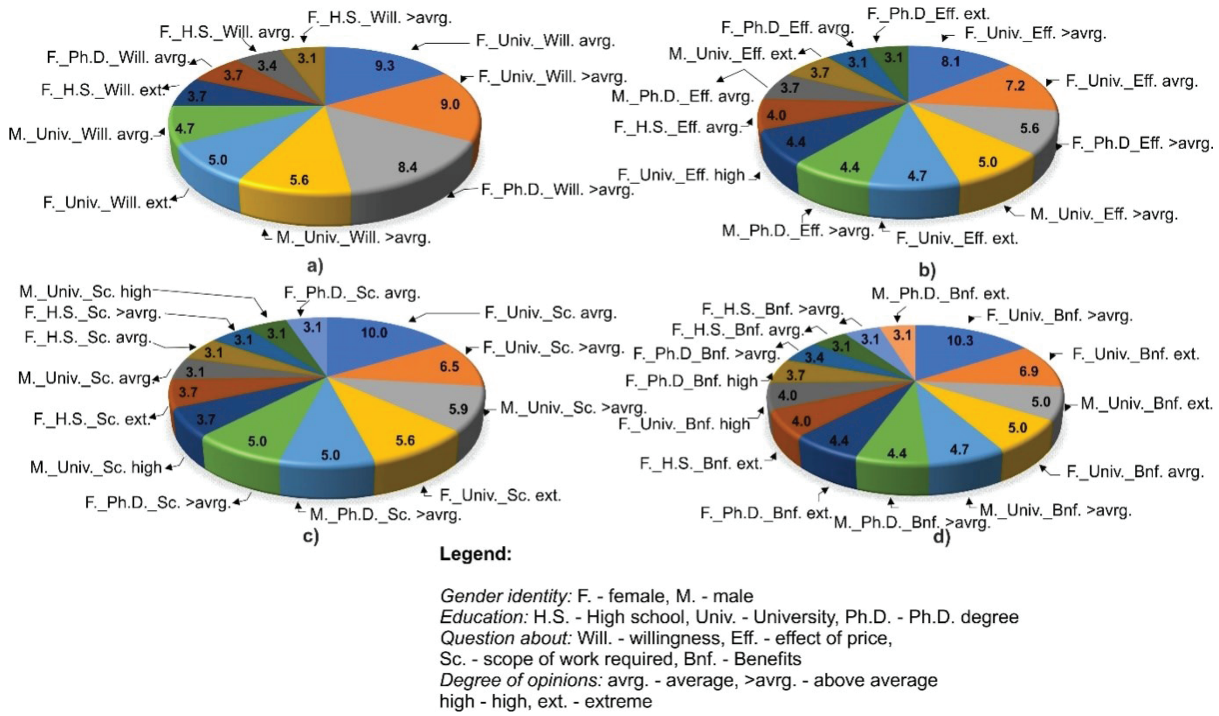

Figure 3. Constructs that show summed links between gender and education of answerers and a) Willingness to use novel materials, b) Effect of price, c) Scope of the required work, and d) Knowledge of the benefits

Finally, the principal factor analysis is implemented to reveal the constructs of answers that have been tested to see the relationships between the parameters observed and the data collected, which fulfill the conclusions of Spearman`s correlation published previously [18]. The constructs were created from the three most frequently appearing questions to show the correlations and the most common combination of responses among them. When grouped (constructed), the gender and education of the participants were the most influential factors in the questions related to novel material usage (Figure 3). Most women who own a university degree have an average willingness to use novel construction and building materials and believe that the scope of work required is of medium importance. These women declared that the price and knowing the benefits of novel materials have significance at an above-average level (Figures 3b and 3d) [15, 24]. A fair number of respondents from the same group (women with a university degree) believed that the scope of work required during the application or installation of novel construction products was extremely important (Fig. 3d). Since more women than men answered the questionnaire, this influenced the number of responses. However, men holding a University degree claimed that their willingness toward novel material usage was above average, and their opinion on the importance of the effects of price and scope of work was at the same level, while the expected benefits were mostly marked as extremely important. Among the Ph.D. holders, most of the four factors were noted as "above average" by men and women, while both also claimed that knowledge of the benefits is of extreme importance, which is consistent with previous studies from developed countries [15]. Most high school-educated women noted an extremely high willingness toward novel material usage and the same level of importance concerning the scope of work and knowledge of the benefits. Other studies revealed that higher educational levels had a positive influence on environmentally friendly choices [15], which is in disagreement with the results of this survey.

4 Conclusions

This preliminary study investigates the effect of socioeconomic issues during the crisis on the use and purchase of construction materials and products in Serbia as an example of a developing country. Furthermore, it offers a broad overview of environmental awareness and consumer acceptance of newly developed sustainable products from the perspective of private individuals. Answerers with higher education degrees were the majority of those who purchased or showed interest in novel products. Only 9% of the respondents use novel materials and among them are mostly carpentry, kitchen work surfaces, specialized paint, and fungal treatment coatings. The willingness to use novel materials is seen as high, but the purchasers would like to gain more information on the prices involved, the scope of work required, and the benefits. The study aims to provide an in-depth perception of green consumer behavior that may aid academics and marketers in better comprehending the issue. To improve the coherence of our understanding of the factors that influence purchases, future studies should also include the respondents' personal and attitude factors, such as flexibility, self-confidence, risk perception and behavior, readiness to act, etc. Besides, having more answerers would be beneficial.

Appendices

A study on the use of building materials in developing countries before and after the pandemic- A socio-economic analysis (List of the questions [18])

Q1. The method of responding to the survey:
- Smartphone
- Desktop
- Tablet
- In paper

Q2. What is your age group?
- 18-30
- 30-40
- 40-50
- 50-60
- 60-70
- 70-80

Q3. What gender are you?
- Male
- Female
- None of the above

Q4. How many inhabitants are there in the place where you live?
- Under 100,000
- Between 100,000 and 300,000
- Between 300,000 - 500,000
- Between 500,000 and 800,000
- Between 800,000 - 1,000,000
- Between 1,000,000 and 2,000,000
- Over 2,000,000

Q5. What is your final education level?
- Primary school
- High School
- College
- Researcher/Doctor of Science
- Professor

Q6. What is your profession/job description?
- Unemployed
- Manual worker
- Office work
- Laboratory technician
- Medical worker
- Craftsman
- Student
- Artist
- Engineer
- Manager
- Scientist
- Professor
- Retired
- Other

Q7. How satisfied are you with your salary concerning the work you do? (the optional question)
From very dissatisfied to very satisfied
(Scale 1-7)

Q8. Has your income changed since the crisis (pandemic) began?
- Incomes have decreased
- They haven't changed
- Incomes have increased
- Not applicable (retired, non-employed)

Q9. When was the last time you changed your place of residence?
- In the last 5 years
-. During the last 10 years
- 20 years ago
- 30 years ago
- More than 40 years ago
- Never

Q10. Do you live in an apartment or a house?
- Apartment
-A house

Q11. Are you renting the space you live in or is it owned by you or your family?
- I'm renting
- I live in mine/our apartment/house

Q12. When was the building/house (you currently live in) built?
- In the last 5 years
-. During the last 10 years
- 20 years ago
- 30 years ago
- 40 or more years ago

Q13. Do you own a cottage, a rest private house, or more than one apartment?
- Yes
- No

Q14. If you own a cottage, a rest private house, or more than one apartment, when was it built?
- In the last 5 years
- During the last 10 years
- 20 years ago
- 30 years ago
- 40 years ago, or more
- Not applicable

Q15. To what extent do you believe that the price of a product speaks of its quality?
(Likert`s scale 1 – 7)
1– very low, 7 – extremely high

Q16. In the period ***before the pandemic****, did you buy construction materials or products (glue, varnish, paint, wall paint, cement, ceramic tiles, sanitary equipment, bricks, tiles, and floor coverings)?*
- Yes, about 2-5 years ago.
- Yes, about 5-10 years ago.
- Yes, over about 10-20 years.
- No

Q17. Based on what did you choose for construction materials in the period ***before the pandemic*** *(glue, varnish, paint, wall paint, cement, etc.)?*
- You choose to buy the most expensive product
- You choose to buy a product whose price is average
- You choose a fair relationship between quality and price
- You choose the cheapest
- You listen to the recommendation of a contractor or a friend/acquaintance you trust
- You buy from familiar manufacturers
- I did not buy construction material during that period

Q18. Based on what did you choose for construction products in the period ***before the pandemic*** *(ceramic tiles, sanitary equipment, bricks, tiles, floor coverings, etc.)?*
- You choose to buy the most expensive product

- You choose to buy a product whose price is average
- You choose a fair relationship between quality and price
- You choose the cheapest
- You listen to the recommendation of a contractor or a friend/acquaintance you trust
- You buy from familiar manufacturers
- I did not buy construction material during that period

*Q19. In the period **after the beginning of the pandemic** (March 2020), did you buy glue, varnish, paint, wall paint, cement, ceramic tiles, sanitary equipment, bricks, tiles, and floor coverings?*
- Yes
- No

*Q20. Based on what did you choose for construction materials (glue, varnish, paint, wall paint, cement, etc.) in the period **after the beginning of the pandemic** (March 2020)?*
- You choose to buy the most expensive product
- You choose to buy a product whose price is average
- You choose a fair relationship between quality and price
- You choose the cheapest
- You pay attention to the recommendation of a contractor or a friend/acquaintance you trust
- You buy from familiar manufacturers
- I did not buy construction material during that period

*Q21. Based on what did you choose for construction products in the period **during the 2020 crisis** (ceramic tiles, sanitary equipment, bricks, tiles, floor coverings, etc.)?*
- You choose to buy the most expensive product
- You choose to buy a product whose price is average
- You choose a fair relationship between quality and price
- You choose the cheapest
- You listen to the recommendation of a contractor or a friend/acquaintance you trust
- You buy from familiar manufacturers
- I did not buy construction material during that period

Q22. To what extent has the way you choose products according to the price/quality ratio changed since the 2020 crisis?
(Likert`s scale of 1-7)
1– very low, 7 – extremely high

Q23. If you bought construction material and/or products in the period before and after the 2020 crisis, to what extent do you have the impression that prices have changed?
(Likert`s scale of 1-7)
1– very low, 7 – extremely high

Q24. To what extent are you willing to accept a newer type of product compared to those traditionally used (nanocoating, cement-based geopolymers, fly ash-based cement, concrete based on geopolymers, self-healing concrete, concrete block instead of brick, concrete reinforced with bamboo, lightweight block of large dimensions, ceramic tiles of large dimensions, etc.)?
(Likert`s scale of 1-7)
1– very low, 7 – extremely high

Q25. To what extent would the price affect the acceptability of switching to some type of better environmental material/product in your household?
(Likert`s scale of 1-7)
1– very low, 7 – extremely high

Q26. To what extent would the scope of work be required to affect the acceptability of switching to some type of better environmental material/product in your household?
(Likert`s scale of 1-7)
1– very low, 7 – extremely high

Q27. To what extent would adequate knowledge of the benefits of new environmental materials/products affect the transition to that material/product in your household?
(Likert`s scale of 1-7)
1– very low, 7 – extremely high

Q28. Do you use any of the innovative materials/products from this sector and which ones?
- Indicate: ____________
- I do not use

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Technical paper

Domain reduction method: formulation, possibilities, and examples for analyzing seismic soil-structure interaction

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ABSTRACT

The Domain Reduction Method (DRM) enables the analysis of seismic soil-structure interaction in a different way compared to other seismic methods. Additionally, it allows certain very important aspects of the mentioned interaction, which are ignored in the usual seismic methods for justified reasons, to be addressed. All this, as well as the fact that this method is still unknown to the professional public and has not been implemented in modern seismic standards, motivated the writing of a paper in which the formulation of the DRM was first presented in detail. Then, the possibilities and approaches to its application in engineering practice were analyzed. In the end, simple dynamic analyses of the seismic interaction of the foundation soil and the pile-supported structure are performed using the DRM, a very specific and insufficiently researched type of seismic soil-structure interaction. Among other things, the results of the performed linear-elastic analyses point to the eventual possibility that the Lateral force seismic method, which is recommended by the Eurocode 8 standard for regular structures and which is most often used in engineering practice, underestimates the level of the lateral seismic load of pile-supported structures. A correct assessment of the seismic load is a fundamental requirement for ensuring a sufficient level of seismic resistance in structures.

1 Introduction

This paper presents a novel method for analyzing seismic soil-structure interaction, which is still relatively unknown among professionals and the scientific community. Known as the Domain Reduction Method (DRM), it was formulated approximately 20 years ago by Bielak et al. [1] and brought about a significant innovation from conventional seismic methods, making it a true "small' revolution. This is also the reason why the DRM encountered disputes at the beginning. Over time, these disputes become less intense, and this seismic method is more and more accepted by the professional and scientific public. However, regardless of the numerous advantages of the DRM, it is still much less frequently used compared to other seismic methods. The formulation of the DRM presented in this paper is taken from: Bielak et al. [1], Youshimura et al. [2], Kantoe et al. [3], and Jeremic et al. [4]. After the formulation of the DRM, the possibilities and ways of its application in engineering practice will be analyzed. Finally, some simple examples of the application of the DRM are presented.

1.1 Development and application of the DRM

The DRM enables the formation and processing of complex seismological (geophysical) 3D numerical models that contain the earthquake source (i.e., fault), wave propagation paths, and local geological and topographical structures [2]. Also, this method enables the formation complex 3D numerical soil-structure models and simulating real seismic excitation in these models, although they don't contain earthquake source and wave propagation paths. For this reason, the DRM is very suitable for the analysis of dynamic (seismic) soil-structure interactions. Essentially, the dimensions of the foundation soil domain during the formation of numerical soil-structure models are reduced by changing the governing variables [3].

The basic idea of the DRM, which implies the reduction of the dimensions of the soil domain by replacing the governing variables with the assumption that the ground motion during an earthquake in the absence of the structure (free-field ground motion) is known, was presented for the first time by Hererra & Bielak [5]. Also, they proposed an analytical solution on how to determine the displacements of the structure and the soil around the structure during an earthquake based on the known free-field ground motion during an earthquake. The procedures for solving this problem using the finite element method were defined by Bielak and Christiano [6]. In both procedures, seismic excitation is replaced by effective seismic forces applied along the contours of the reduced soil domain. However, the problem with these procedures is that they required the

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determination of unknown effective seismic forces in order to define free-field ground motion. Bielak et al. [1] suggested the two-step procedure as a solution to this problem. In this way, the final formulation of the DRM, which is presented in this paper, was obtained. Kontoe et al. [3] suggested the formulation of the DRM for dynamic coupled consolidation analysis.

Regardless of its quality and reliability, the DRM is rarely used in engineering practice. There are two main reasons for this. Firstly, the DRM is still not "recognized" by seismic standards. The exception is the ASCE/SEI 4-16 [7] standard, which proposes the application of this method in seismic resistance analyses of nuclear ficilities. There are several works on this topic in the professional and scientific literature [8-12]. Secondly, the DRM has not yet been implemented in the software most commonly used in engineering practice for the design of structures.

1.2 A brief overeview of methods for seismic soil-structure interaction analysis

In the middle of the last century, it became clear to engineers that in order to assess the real seismic response of a structure, it is necessary to analyze the interaction of that structure and the foundation soil during an earthquake. Since then, several methods have been developed for the analysis of seismic soil-structure interaction (hereinafter seismic SSI methods). Their development coincides with the development of software that enables more complicated and demanding analysis of structures using numerical methods.

The seismic SSI methods can be divided in several ways. Depending on the method of soil modelling, seismic SSI methods with discrete and continuum soil modelling are distinguished. In discrete soil modelling, the soil is replaced by a series of springs, or springs and daspots (rheological elements). In continuum soil modelling, appropriate finite and/or boundary elements are most often used. Depending on the material characteristics of the elements used in soil or structural element modelling, the seismic SSI methods can be linear or nonlinear. Usually, only material nonlinearity is considered. However, in situations with intensive yielding of structural elements and/or with intensive yielding of soil (liquefaction), material and geometric nonlinearity must be taken into account. Some seismic SSI methods involve solving the equation of motion of the soil-structure system in the frequency domain. Other seismic SSI methods involve solving the equation of motion of the soil-structure system in the time domain. Methods that use the frequency domain are simpler, but they are not suitable for analyzing the nonlinear behavior of the soil-structure system during an earthquake. A special group consists of the so-called hybrid methods that use both domains.

All seismic SSI methods are based on two main approaches. These are direct and substructure approaches. For this reason, we can talk about direct and substructure seismic SSI methods. In the direct seismic SSI method ("one-step" method), the equation of motion of the complete soil-structure system is solved at once (in one step), usually with free-field ground motion as the input load of the system and usually in the time domain. In the substructure seismic SSI method ("two-step" method), the equation of motion of the system, which contains only the so-called substructure (soil and structural foundation) with free-field ground motion as an input load, is solved first (kinematic interaction) in order to obtain displacements (accelarations, velocities) of the structural foundation during an earthquake. In this case, some methods take into account the real stiffness of the structural foundation. Other methods assume that the structural foundation is rigid. In the second step, the equation of motion of the system, which contains the superstructure, springs, and dashpots (or more complex nonlinear elements to represent stiffness and damping of the substructure), is solved (inertial interaction) with the previously determined seismic response of the structural foundation as the input load. Frequency-dependent stiffness and damping of the substructure represent so-called dynamic impedances of the foundation. Finally, the results of kinematic and inertial interactions are superimposed.

All previously mentioned seismic SSI methods are also applicable in analyzes of the interaction between the soil and the pile-supported structure during an earthquake (seismic SPS interaction). Piles are usually modelled using a beam of finite elements. The contact between the surrounding soil and the piles is simulated using discrete rheological elements (springs, springs, and daspots, etc.) or interface finite elements. In the substructure seismic SSI method, the first step involves solving the equation of motion of the system, which consists of soil, piles, and pile caps. In the second step, it is necessary to define the dynamic impedances of the pile foundation.

2 Formulation of the domain reduction method

Fig. 1a shows a very simplified engineering, and seismo-geological model of the region of interest. A simple fault that represents a potential source of an earthquake is also modeled. Models of this type are usually kilometers in size, and seismologists use them to analyze the seismic hazards of the region of interest. The seismic excitation of the region of interest, considering the assumed fault type and characteristics, is defined by analyzing the generated model, typically in the form of displacement, velocity, and acceleration fields. So, in this way, the seismic excitation in the zone of any object (structure) within the treated region is defined. However, it is impossible to analyze the soil-structure interaction for any object during an earthquake on a model of these dimensions. In order to analyze this interaction, the question arises as to how only a smaller zone of soil around the object of interest can be separated from the formed and processed seismo-geological model, but in such a way that the previously defined seismic excitation remains ''trapped'' in it (see Fig. 1b). The solution is given by Bielak et al. [1], who formulate the DRM.

Γ marks the boundary between the outside soil subdomain Ω^{+} and the inside soil subdomain Ω. This boundary is taken into account in the seismic soil-structure interaction analysis for the object of interest. The dimensions of the inside domain are usually 3-4 times larger than the dimensions of the object. Nodal displacements of the outside subdomain Ω^{+}, inside subdomain Ω, and boundary between them Γ are denoted by u_e, u_i and u_b respectively. So, the subscripts i, e and b refer to the part of the analysed soil domain to which some quantity refers. For the analysed soil domain, the equation of motion in the case of forced undamped oscillations can be expressed in matrix form as:

$$\mathbf{M} \cdot \ddot{\mathbf{u}} + \mathbf{K} \cdot \mathbf{u} = \mathbf{P_e} \tag{1}$$

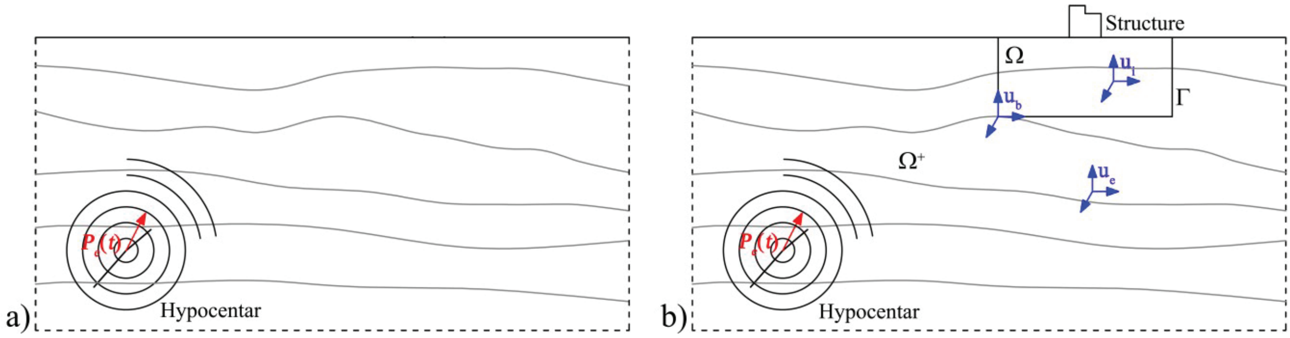

Figure 1. a) The seismo-geological model of the region of interest with a potential source of earthquake b) Division of the seismo-geological model domain into two subdomains. (Adapted from [1])

or can be expressed in partitioned form for both soil subdomains as:

$$\begin{bmatrix} \mathbf{M}_{ii}^{\Omega} & \mathbf{M}_{ib}^{\Omega} & 0 \\ \mathbf{M}_{bi}^{\Omega} & \mathbf{M}_{bb}^{\Omega}+\mathbf{M}_{bb}^{\Omega^+} & \mathbf{M}_{be}^{\Omega^+} \\ 0 & \mathbf{M}_{eb}^{\Omega^+} & \mathbf{M}_{ee}^{\Omega^+} \end{bmatrix} \cdot \begin{Bmatrix} \ddot{\mathbf{u}}_i \\ \ddot{\mathbf{u}}_b \\ \ddot{\mathbf{u}}_e \end{Bmatrix} + \begin{bmatrix} \mathbf{K}_{ii}^{\Omega} & \mathbf{K}_{ib}^{\Omega} & 0 \\ \mathbf{K}_{bi}^{\Omega} & \mathbf{K}_{bb}^{\Omega}+\mathbf{K}_{bb}^{\Omega^+} & \mathbf{K}_{be}^{\Omega^+} \\ 0 & \mathbf{K}_{eb}^{\Omega^+} & \mathbf{K}_{ee}^{\Omega^+} \end{bmatrix} \cdot \begin{Bmatrix} \mathbf{u}_i \\ \mathbf{u}_b \\ \mathbf{u}_e \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \\ \mathbf{P}_e \end{Bmatrix} \tag{2}$$

On the left side of Eq. (2), the matrices $\mathbf{M}$ and $\mathbf{K}$ denote the mass and stiffness submatrices, the vectors $\ddot{\mathbf{u}}$ and $\mathbf{u}$ denote the nodal accelerations and displacements subvectors. On the right side of Eq. (2), vector $\mathbf{P}_e$ denotes the subvector of unknown seismic nodal forces. The outside soil subdomain Ω^+ and the inside soil subdomain Ω with the object of interest can be separated from each other. Therefore, the above equation can be simply divided into two equations as follows:

But it is very important to remember that the inside and outside soil subdomains and their equations of motion are separated based on the assumption that nodal displacements u_b and nodal forces P_b are compatible along the boundary between the two soil subdomains (see Fig. 2a).

$$\begin{bmatrix} \mathbf{M}_{ii}^{\Omega} & \mathbf{M}_{ib}^{\Omega} \\ \mathbf{M}_{bi}^{\Omega} & \mathbf{M}_{bb}^{\Omega} \end{bmatrix} \cdot \begin{Bmatrix} \ddot{\mathbf{u}}_i \\ \ddot{\mathbf{u}}_b \end{Bmatrix} + \begin{bmatrix} \mathbf{K}_{ii}^{\Omega} & \mathbf{K}_{ib}^{\Omega} \\ \mathbf{K}_{bi}^{\Omega} & \mathbf{K}_{bb}^{\Omega} \end{bmatrix} \cdot \begin{Bmatrix} \mathbf{u}_i \\ \mathbf{u}_b \end{Bmatrix} = \begin{Bmatrix} 0 \\ \mathbf{P}_b \end{Bmatrix} \quad \text{for inside soil subdomain } \Omega \tag{3}$$

$$\begin{bmatrix} \mathbf{M}_{bb}^{\Omega^+} & \mathbf{M}_{be}^{\Omega^+} \\ \mathbf{M}_{eb}^{\Omega^+} & \mathbf{M}_{ee}^{\Omega^+} \end{bmatrix} \cdot \begin{Bmatrix} \ddot{\mathbf{u}}_b \\ \ddot{\mathbf{u}}_e \end{Bmatrix} + \begin{bmatrix} \mathbf{K}_{bb}^{\Omega^+} & \mathbf{K}_{be}^{\Omega^+} \\ \mathbf{K}_{eb}^{\Omega^+} & \mathbf{K}_{ee}^{\Omega^+} \end{bmatrix} \cdot \begin{Bmatrix} \mathbf{u}_b \\ \mathbf{u}_e \end{Bmatrix} = \begin{Bmatrix} -\mathbf{P}_b \\ \mathbf{P}_e \end{Bmatrix} \quad \text{for outside soil subdomain } \Omega^+ \tag{4}$$

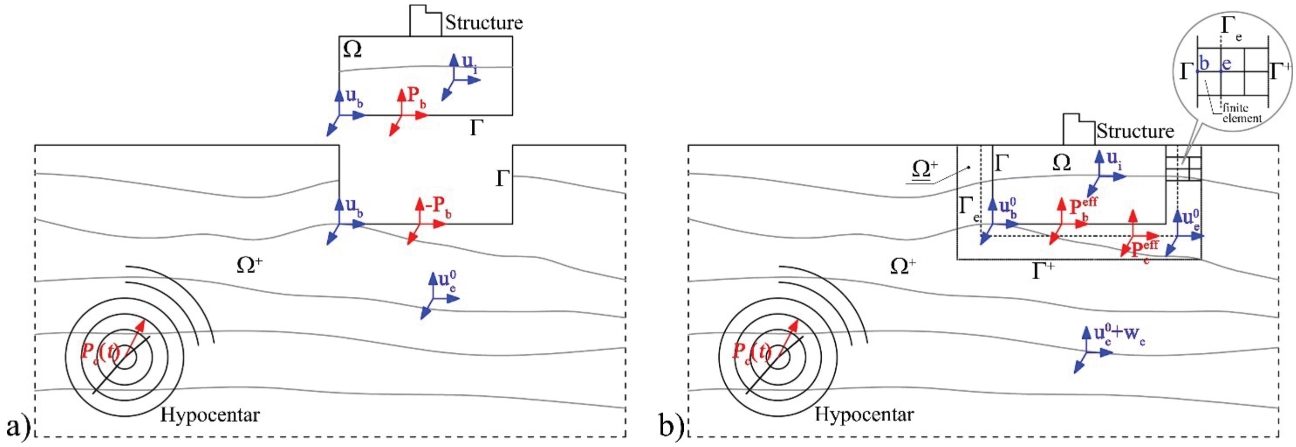

Figure 2. a) Separated inside and outside soil subdomain of the analysed seismo-geological model b) Dimension reduction of the outside soil subdomain, DRM and Damping layer

Eq. (4) is used for the elimination of the unknown values of the seismic nodal forces P_e from Eq. (2). Since Eq. (4) is valid for the separated outside soil subdomain Ω^+, so there is no influence of the inside subdomain and the object, the values of outside nodal displacements u_e and accelerations $\ddot{u}_e$ actually correspond to the values of outside nodal displacements and accelerations calculated for the free-field soil model (free-field outside nodal displacements and accelerations) u_e^0 and $\ddot{u}_e^0$. In that case, Eq. (4) can be written as:

$$\begin{bmatrix} \mathbf{M}_{bb}^{\Omega^+} & \mathbf{M}_{be}^{\Omega^+} \\ \mathbf{M}_{eb}^{\Omega^+} & \mathbf{M}_{ee}^{\Omega^+} \end{bmatrix} \cdot \begin{Bmatrix} \ddot{\mathbf{u}}_b^0 \\ \ddot{\mathbf{u}}_e^0 \end{Bmatrix} + \begin{bmatrix} \mathbf{K}_{bb}^{\Omega^+} & \mathbf{K}_{be}^{\Omega^+} \\ \mathbf{K}_{eb}^{\Omega^+} & \mathbf{K}_{ee}^{\Omega^+} \end{bmatrix} \cdot \begin{Bmatrix} \mathbf{u}_b^0 \\ \mathbf{u}_e^0 \end{Bmatrix} = \begin{Bmatrix} -\mathbf{P}_b^0 \\ \mathbf{P}_e \end{Bmatrix} \tag{5}$$

The free-field outside nodal displacements and accelerations are already known because of how the free-field seismo-geological model shown in Fig. 1a was made and how it was used. Therefore, the above equation can be used to calculate the unknown seismic nodal forces P_e. These forces are equal to:

$$\mathbf{P_e} = \mathbf{M}_{eb}^{\Omega^+} \cdot \ddot{\mathbf{u}}_b^0 + \mathbf{M}_{ee}^{\Omega^+} \cdot \ddot{\mathbf{u}}_e^0 + \mathbf{K}_{eb}^{\Omega^+} \cdot \mathbf{u}_b^0 + \mathbf{K}_{ee}^{\Omega^+} \cdot \mathbf{u}_e^0 \tag{6}$$

According to the main assumption and transformation of the DRM, the displacement of any node in the outside soil subdomain Ω^+ can be expressed in the form of the following sum of displacements:

$$\mathbf{u_e} = \mathbf{u}_e^0 + \mathbf{w_e} \tag{7}$$

where $\mathbf{w}_e$ represent a vector of "residual" displacement field, i.e., a vector of relative displacement field with respect to the reference vector of free field displacement $\mathbf{u}_e^0$. Actually, in the above equation, in terms of the vector $\mathbf{w}_e$ represent the changes in the free-field outside nodal displacements caused by the oscillation of the object (structure) during an earthquake. After substituting Eq. (7) into Eq. (2), the equation of motion for the analyzed soil domain can be written as:

$$\begin{bmatrix} \mathbf{M}_{ii}^{\Omega} & \mathbf{M}_{ib}^{\Omega} & 0 \\ \mathbf{M}_{bi}^{\Omega} & \mathbf{M}_{bb}^{\Omega} + \mathbf{M}_{bb}^{\Omega^+} & \mathbf{M}_{be}^{\Omega^+} \\ 0 & \mathbf{M}_{eb}^{\Omega^+} & \mathbf{M}_{ee}^{\Omega^+} \end{bmatrix} \cdot \begin{Bmatrix} \ddot{\mathbf{u}}_i \\ \ddot{\mathbf{u}}_b \\ \ddot{\mathbf{u}}_e^0 + \ddot{\mathbf{w}}_e \end{Bmatrix} + \begin{bmatrix} \mathbf{K}_{ii}^{\Omega} & \mathbf{K}_{ib}^{\Omega} & 0 \\ \mathbf{K}_{bi}^{\Omega} & \mathbf{K}_{bb}^{\Omega} + \mathbf{K}_{bb}^{\Omega^+} & \mathbf{K}_{be}^{\Omega^+} \\ 0 & \mathbf{K}_{eb}^{\Omega^+} & \mathbf{K}_{ee}^{\Omega^+} \end{bmatrix} \cdot \begin{Bmatrix} \mathbf{u}_i \\ \mathbf{u}_b \\ \mathbf{u}_e^0 + \mathbf{w}_e \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \\ \mathbf{P}_e \end{Bmatrix} \tag{8}$$

As stated previously, the values of free-field outside nodal displacements and accelerations (u_e^0 and $\ddot{u}_e^0$) are known. Therefore, they can be moved to the right side of the above equation. Lastly, substituting Eq. (6) into Eq. (8), the equation of motion for the analyzed soil domain in the case of forced undamped oscillations can be written as:

$$\begin{bmatrix} \mathbf{M}_{ii}^{\Omega} & \mathbf{M}_{ib}^{\Omega} & 0 \\ \mathbf{M}_{bi}^{\Omega} & \mathbf{M}_{bb}^{\Omega} + \mathbf{M}_{bb}^{\Omega^+} & \mathbf{M}_{be}^{\Omega^+} \\ 0 & \mathbf{M}_{eb}^{\Omega^+} & \mathbf{M}_{ee}^{\Omega^+} \end{bmatrix} \cdot \begin{Bmatrix} \ddot{\mathbf{u}}_i \\ \ddot{\mathbf{u}}_b \\ \ddot{\mathbf{w}}_e \end{Bmatrix} + \begin{bmatrix} \mathbf{K}_{ii}^{\Omega} & \mathbf{K}_{ib}^{\Omega} & 0 \\ \mathbf{K}_{bi}^{\Omega} & \mathbf{K}_{bb}^{\Omega} + \mathbf{K}_{bb}^{\Omega^+} & \mathbf{K}_{be}^{\Omega^+} \\ 0 & \mathbf{K}_{eb}^{\Omega^+} & \mathbf{K}_{ee}^{\Omega^+} \end{bmatrix} \cdot \begin{Bmatrix} \mathbf{u}_i \\ \mathbf{u}_b \\ \mathbf{w}_e \end{Bmatrix} = \begin{Bmatrix} 0 \\ -\mathbf{M}_{be}^{\Omega^+} \cdot \ddot{\mathbf{u}}_e^0 - \mathbf{K}_{be}^{\Omega^+} \cdot \mathbf{u}_e^0 \\ \mathbf{M}_{eb}^{\Omega^+} \cdot \ddot{\mathbf{u}}_b^0 + \mathbf{K}_{eb}^{\Omega^+} \cdot \mathbf{u}_b^0 \end{Bmatrix} \tag{9}$$

It is already known what the terms of the above equation represent. All unknowns are on the left side of the equation. The vector on the right side of the equation represents the seismic effective nodal force vector. This vector can be written as:

$$\mathbf{P}^{\mathbf{eff}} = \begin{Bmatrix} \mathbf{P}_i^{\mathbf{eff}} \\ \mathbf{P}_b^{\mathbf{eff}} \\ \mathbf{P}_e^{\mathbf{eff}} \end{Bmatrix} = \begin{Bmatrix} 0 \\ -\mathbf{M}_{be}^{\Omega^+} \cdot \ddot{\mathbf{u}}_e^0 - \mathbf{K}_{be}^{\Omega^+} \cdot \mathbf{u}_e^0 \\ \mathbf{M}_{eb}^{\Omega^+} \cdot \ddot{\mathbf{u}}_b^0 + \mathbf{K}_{eb}^{\Omega^+} \cdot \mathbf{u}_b^0 \end{Bmatrix} \tag{10}$$

The seismo-geological model (see Fig. 1a) shows that the seismic effective nodal forces P^{eff} always replace the seismic nodal forces P_e that are made by the fault crack. Consequently, using seismic effective nodal forces P^{eff}, all real seismic waves (P, SV, SH, Rayleigh and Love waves) can be adequately modelled. According to Eq. (10), in order to define the forces P^{eff}, the values of free-field nodal accelerations and displacements ($\ddot{u}^0$ and u^0) are necessary. However, it is very interesting and significant to state that in Eq. (10) these accelerations and displacements are multiplied only with the mass and stiffness matrices of those finite elements of the outside soil subdomain Ω^+ that are located along the boundary Γ, i.e., between the boundaries Γ and its adjacent boundary Γ_e (see Fig. 2b). The boundary Γ_e represents an outside contour (surface) of the fictitious soil layer, which will be discussed a little later and is particularly important in the DRM. Therefore, the seismic effective nodal forces P^{eff} act only on the nodes of the finite elements of the outside soil subdomain located between the boundaries Γ and Γ_e. For this reason, in order to determine the intensity of forces P^{eff}, it is necessary to know the values of accelerations $\ddot{u}_0$ and displacements u_0 only for those nodes. In order to simplify the calculation, i.e., reduce the number of unknown effective nodal forces P^{eff} that must be determined, it can be assumed that the boundaries Γ and Γ_e are close enough to each other, so there is only one layer of finite elements between them. In this situation, the seismic effective forces P^{eff} act only on the nodes of that one layer of finite elements. This layer of finite elements is called the DRM layer. This localization of forces P^{eff} is a direct consequence of the outside subdomain nodal displacement transformation, i.e., a direct consequence of Eq. (7).

In addition to the previously described method of defining the seismic load, the DRM's expression of the equation of motion (Eq. (9)). As can be seen, the unknowns in this equation are only displacements of the nodes on the boundaries Γ and Γ_e (see Fig. 2b). For the outside soil subdomain Ω^+ i.e., part of the soil beyond the boundary Γ_e, the "residual" displacement field w_e is obtained by solving Eq. (9). The "residual" displacement field w_e is relative displacement field with respect to the primary displacement field u^0 (free-field displacements). In the soil-structure interaction analyses, attention is focused on the structure and the foundation soil. For this reason, this "residual" displacement field has no practical significance. This fact, as well as the previously described way of defining the seismic load in the DRM, allows a drastic reduction of the complete outside soil subdomain to a smaller soil subdomain $\underline{\Omega}^+$ around the DRM layer, i.e., the subdomain between the boundaries Γ_e and Γ^+ (see Fig. 2b). Due to the possibility of reducing the dimensions of the soil domain, this seismic method is called the Domain Reduction Method (DRM). A sufficiently high damping level should be adopted for the material of the reduced outside soil subdomain $\underline{\Omega}^+$, in order to prevent the occurrence of spurious seismic waves. These waves can be generated by waves from the inside soil subdomain passing through the outside soil subdomain, hitting the model boundary, and being reflected back to the inside soil subdomain and structure. The reduced outside soil subdomain $\underline{\Omega}^+$ with pronounced material damping is called the Damping layer and is modeled with two or more layers of finite elements.

Two very significant facts of the DRM that refer to the reduced outside soil subdomain $\underline{\Omega}^+$ i.e., the Damping layer, should be mentioned. First, according to the main assumption, i.e., the transformation of the DRM (Eq. (7)), outside subdomain nodal displacements are obtained by applying the principle of superposition. In most cases, the main reason for disputing the DRM by the professional and scientific public was related to this transformation. It is generally known that the principle of superposition can be applied only in the case of elastic materials, i.e., only in the case of linear-elastic analyses. However, the disputed principle of superposition (Eq. (7)) is only valid for the outside soil subdomain $\underline{\Omega}^+$. Therefore, it does not apply to the structure, and it does not apply to the foundation soil, i.e., inside soil subdomain Ω. It is only valid for the soil that is located at a sufficiently large distance from the structure, i.e., for the outside soil subdomain $\underline{\Omega}^+$. This soil subdomain is not of interest in the soil-structure interaction analyses. So, engineers, when using the DRM, should not be concerned about the adopted assumption related to the linear-elastic behavior of the outside soil subdomain material.

The previous statement, related to the superposition of outside subdomain nodal displacements, can also be accepted for the high level of damping that is adopted for the outside soil subdomain $\underline{\Omega}^+$ i.e., for the Damping layer. This is the second important fact that should be mentioned. In this way, a very significant problem in the dynamic soil-structure interaction analysis is easily overcome. It is a problem of boundary conditions along the artificial boundaries of the modeled soil domain (model boundaries). In the usual methods of analyzing the seismic soil-structure interaction, viscous dampers, i.e., dashpots, are placed along these boundaries. Their task is to absorb seismic waves that hit the model boundaries. However, these elements only absorb waves that hit the model boundaries at the right angle. So, if the wave hits the model boundary at some oblique angle, the effectiveness of the dashpots is problematic. The adoption of a high level of damping for the Damping layer in the DRM enables the placement of the simplest supports (pins or rollers) along the model boundaries. Also, it enables seismic excitation to be applied to the soil-structure system in any direction. This opens up new possibilities in seismic soil-structure interaction analyses.

Based on what has been said, it can be said that if the DRM is used, it is necessary to define the material properties of two more fake soil layers in addition to the material properties of the inside soil subdomain Ω (inside soil). It is the DRM and Damping layer. Usually, linear-elastic materials with all the same characteristics except damping are adopted for those soil layers. The material of the Damping layer has a high level of damping, while the DRM layer is without damping. All other material characteristics of the DRM and Damping layer are identical, and their values are adopted based on the values of the inside soil material characteristics.

3 Use of the DRM in engineering practice

Regardless of its quality and reliability, the previously described DRM is rarely used in engineering practice. Unfortunately, the DRM has not yet been implemented in the software most commonly used in engineering practice for the design of structures. Therefore, the use of the DRM is mainly related to scientific research in the field of seismic soil-structure interaction.

According to Eq. (10), in order to use DRM to analyze the interaction of foundation soil and structure at some location during an earthquake, it is necessary to define the seismic excitation for that location. Defining the seismic excitation implies the determination of the values of free-field displacements u^0 and accelerations $\ddot{u}^0$ for all nodes of the DRM layer, which are necessary for calculating the intensity of effective seismic forces. Generally, these values are obtained by processing the seismo-geological model of the location of interest, i.e., the wider area to which the location belongs. However, models of this type are very rare. So far, such a seismo-geological model for any part of Montenegro has not been formed. Therefore, some alternative solutions are usually applied. These solutions imply the use of appropriate recordings of previous earthquakes. If there are recordings of free-field ground motions during previous earthquakes for the location (area) where the analyzed object is located, this can be a very favorable circumstance for engineers when implementing the DRM. In these situations, existing unscaled or scaled recordings can be used as input data for the DRM, i.e., as so-called input accelerograms for the DRM. However, if these recordings do not exist for a location of interest, then appropriate scaled or unscaled recordings of previous earthquakes downloaded from one of the many Internet Ground Motion Databases (GMDB) are used as input accelerograms for the DRM. Examples can be singled out: the European Strong-Motion Database (ESD), the Engineering Strong-Motion Database (ESM), and the PEER Ground Motion Database (Pacific Earthquake Engineering Research Center). However obtained, the input accelerograms and their corresponding displacement recordings are used to determine the values of free-field displacements u^0 and accelerations $\ddot{u}^0$ for all nodes of the DRM layer. According to Eq. (10), if these accelerations and displacements are known, the intensities of the effective seismic forces can be calculated, which provides the conditions for the application of the DRM.

Usually, corrected acceleration recordings (and often also velocity and displacement) can be downloaded from the GMBD for a specific earthquake and location (seismograph station) in two horizontal directions that are perpendicular to each other (East-West and North-South) and in the vertical direction. Thus, there are three components of acceleration and three components of displacement. So far, earthquakes with two or only one component of displacement (acceleration) have not been registered. Engineers interpret the downloaded recordings by considering that every ''point'' on or in the soil (hereinafter soil point), i.e., on or in the structure (depending on where the seismograph is placed), was exposed to displacements in the direction of all three axes during an earthquake. However, this interpretation is only partially correct because any point at and near the soil surface (in shallower layers) is also exposed to rotations around all three axes during an earthquake [12-14]. These rotations actually mean different displacements of two adjacent points, i.e., points at a small distance from each other. So, instead of three, there are actually six displacement components. On concrete numerical models, Jeremić et al. [4] showed a significantly different seismic response of the soil in the case when only the previously described componental displacements are taken into account and in the case when both componental displacements and componental rotations are taken into account.

In the DRM, the input accelerograms and their corresponding displacement recordings can be used in different ways to determine the values of accelerations $\ddot{u}^0$ and displacements u^0 of all nodes of the DRM layer. These accelerations and displacements are needed to calculate the effective seismic forces. The simplest way, but also the one that least corresponds to the real situation, implies the complete neglect of the previously mentioned seismic rotations of the soil points (soil particles). Input accelerograms and their corresponding displacement recordings are ''joined'' to the displacement directions of the soil points. After that, the position of the reference point (coordinate z_R) is defined, which is assumed to be the place of registration of the input accelerograms. In the next step, known dynamic methods, which for the soil profile at a given location perform one-dimensional (vertical) propagation of input accelerograms through the soil, are applied. Usually, the linear or possibly equivalent linear method is used. If it is chosen that the reference point is located on the soil surface (z_R=0), which is most often the case, then the 1D deconvolution of input accelerograms through the soil profile is actually performed. If it is chosen that the reference point is located at a certain depth (z_R<0) e.g., the level of the bedrock, then the 1D convolution of input accelerograms through the soil profile is actually performed. Depending on the type of analysis performed, regardless of the position of the reference point, with this 1D propagation of the input accelerogram through the soil profile, for each point of this profil, the values of one, two, or all three components of the acceleration $\ddot{u}^0$ or displacement u^0 during an earthquake are determined. Finally, the calculated accelerations and displacements are "joined" to the corresponding nodes of the DRM layer. Thus, according to Eq. (10), the conditions for calculating the intensity of effective seismic forces and using the DRM are obtained.

If the input accelerograms and their corresponding displacement recordings are used in the previously described manner, one can speak of 1×1C, 2×1C or 3×1C DRM depending on how many components of acceleration $\ddot{u}^0$ and displacement u^0 are taken into account when calculating the intensity of effective seismic forces. The main shortcoming of these analyses is the fact that when determining accelerations $\ddot{u}^0$ and displacements u^0, as necessary data for calculating effective seismic forces, only body seismic waves are taken into account. In other words, the determination of accelerations $\ddot{u}^0$ and displacement u^0 is performed under the assumption that the movement of the soil during an earthquake is the result of the vertical propagation of P and S body seismic waves from the hypocentar to the soil surface. For this reason, identical displacements of all soil points with the same coordinate z during an earthquake are obtained. This does not correspond to the real situation, especially for soil points at a shallower depth whose movements are dominantly influenced by surface seismic waves. It is known that the influence of surface waves on the seismic excitation to which the structure is exposed can be very significant and often dominant. Surface seismic waves, i.e., their destructiveness, come to the fore in shallow earthquakes (hypocentar depth up to 70km), while deep earthquakes do not produce this type of seismic wave (hypocentar depth greater than 300km).

If the previously described variant of the DRM is correctly implemented, for any soil point with the coordinate z=z_R, the input accelerogram and its corresponding displacement recording "joined" to one of the global axes must be identical to the obtained (output) accelerogram and its corresponding displacement recording for that global axis.

Another way in which input accelerograms and their corresponding displacement recordings can be used to calculate the intensity of effective seismic forces is similar to the previous one. The difference is in the adopted direction of seismic wave propagation. Previously, propagation was vertical. Now, it is inclined, i.e., seismic waves propagate from the source of the earthquake to the soil surface at a certain angle θ in relation to the vertical axis (up to 10°, possibly 15°, rarely more). So, this is the case of the inclined convolution of the input accelerogram through the analyzed soil profile. At the beginning, for the purposes of implementing this convolution, it is necessary to define the coordinates of the soil point that is adopted as the source of the seismic excitation (coordinates x_S, y_S, z_S), which are characterized by the adopted accelerogram and its corresponding displacement recording. This point may be within or outside of the boundaries of the numerical model that is formed to implement the DRM. In the described way, in addition to the effects of body seismic waves, the effects of surface seismic waves, which arise as a result of the interaction of the ''inclined'' body waves and the soil surface, are also tried to be taken into account. The effects of surface waves primarily imply the relative displacement of adjacent points of shallower soil layers during an earthquake, i.e., the occurrence of the previously mentioned seismic rotations of the soil points.

In general, 3C or 6C DRM can be used if the input accelerograms and their corresponding displacement records are used in the way already described. This depends on how the seismic excitation is defined and the type of numerical model. More precisely, if a 2D numerical model of the soil-structure system and the previously described way of applying input accelerograms and their corresponding displacement recordings are used, each soil point is simultaneously subjected to two componental displacements (in the direction of the horizontal and vertical axes – mean axes) and to a componental rotation around an axis perpendicular to the plane of the numerical model. In this case, it is about 3C DRM. If a 3D numerical model of the soil-structure system and the previously described way of applying input accelerograms and their corresponding displacement recordings are used, each soil point is simultaneously subjected to three componental displacements (in the direction of two horizontal and vertical axes – mean axes) and to componental rotations around all mean axes. In this case, it is about 6C DRM. As already mentioned, these componental rotations actually represent the relative displacements of adjacent soil points during an earthquake.

In the end, it is important to note that the way of simulating seismic excitation in the DRM enables the emergence of surface seismic waves as a "result" of the interaction of inclined body waves with appropriate characteristics (frequency and angle θ relative to vertical axes) and shallower layers of foundation soil [15]. No evidence was found in the professional and scientific literature that this way of simulating surface seismic waves is applicable to other seismic SSI methods.

4 Numerical example

4.1 Input data

The previously formulated DRM is presented and demonstrated on the simple example of the interaction of a two-dimensional (2D) RC pile-supported frame and layered foundation soil during an earthquake (see Fig. 3). For the analyzed system, which consists of the soil, pile foundation, and structure (frame), the term SPS system is used in the following text. Two linear-elastic dynamic analyses are carried out. The first is a 1×1C DRM with vertical, linear convolution of the input accelerogram from the bedrock level (z_R=-17m) to the soil surface. The second, 3C DRM, has a linear, inclined convolution of the input accelerogram (θ=10°) from the soil point with coordinates x_S=0, y_S=0 and z_S=-17m to the soil surface. Fig. 4 shows the input accelerogram used in the dynamic analyses. It was downloaded from the ESM. The horizontal seismic excitation at the level of the base of the RC frame, i.e., at the level of the pile cap beam, was first

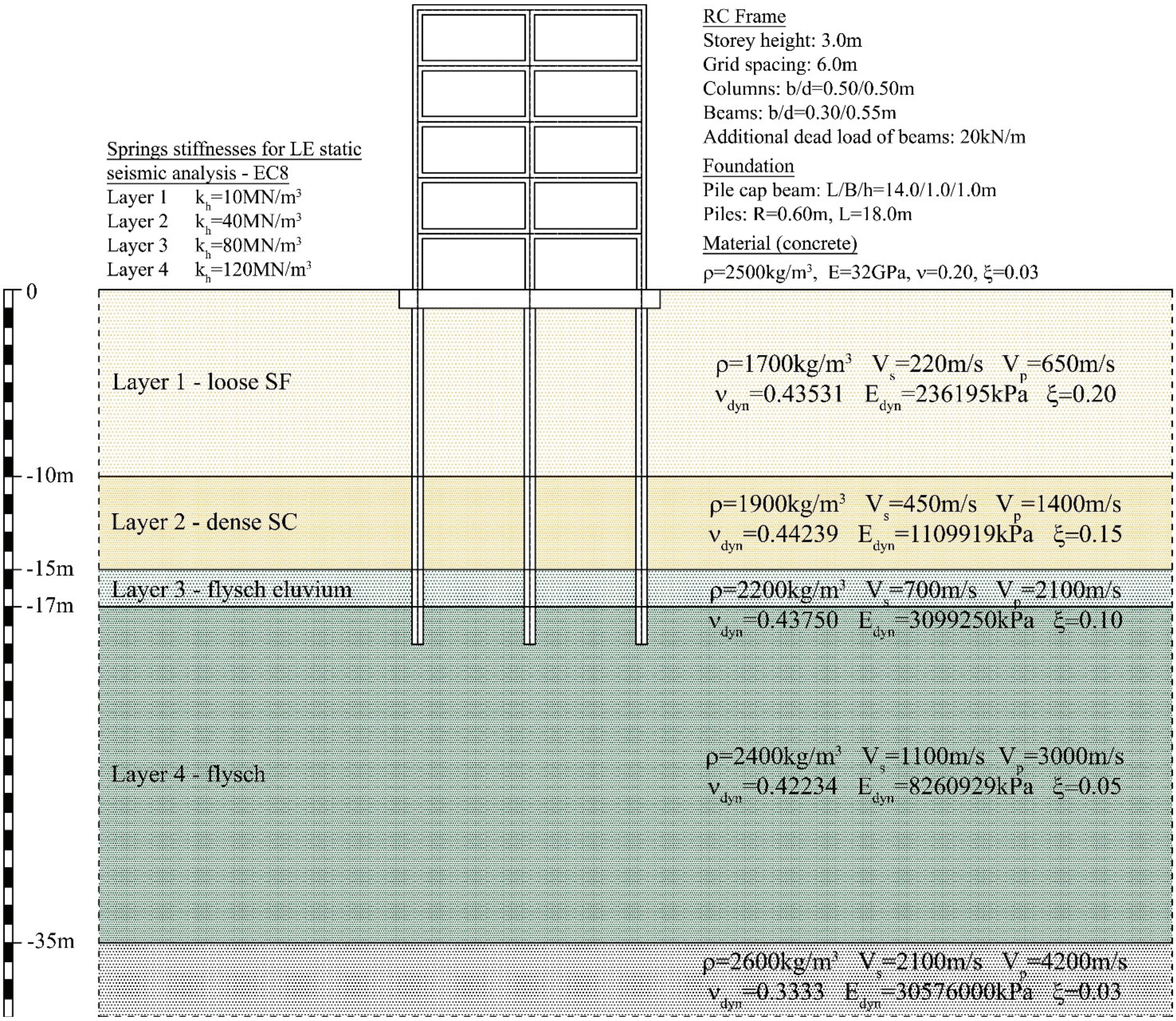

Figure 3. Analysed SPS system with charachteristics of RC frame, foundation and all layers of the foundation soil

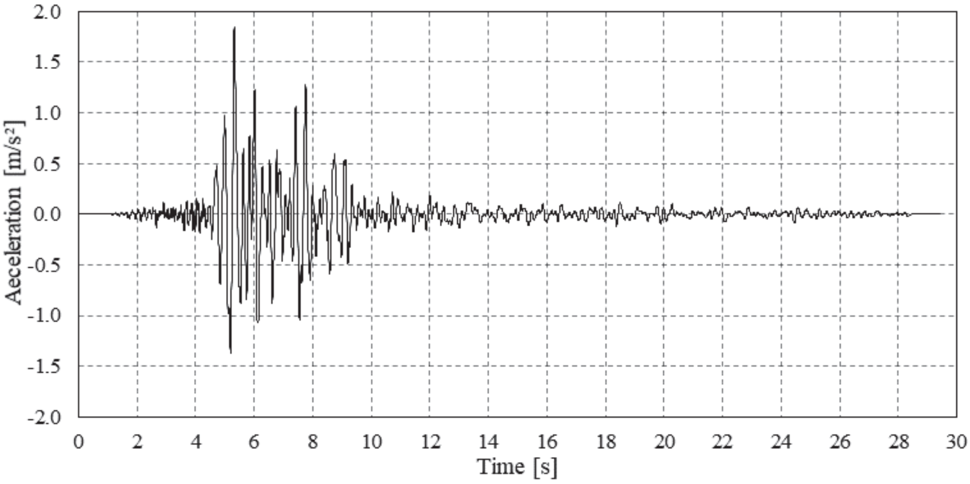

Figure 4. Used accelerogram (26.09.1997., Lower square of St. Francis, comune of Assisi, province of Perugia, region Umbria, Italy, direction East-West, M_w=6.0, epicentral distance 21.6km)

determined by using the DRM. After that, it is compared with the given (input) horizontal seismic excitation at the bedrock level. So, the influence of the upper, soft layers of the foundation soil on the seismic response of the RC pile-supported frame is analyzed.

In the end, it was considered highly useful to carry out a linear-elastic, static seismic analysis of the RC pile-supported frame in a manner common (well-known and generally accepted) in engineering practice. The elastic lateral seismic load of the analyzed frame was determined according to the standard Eurocode 8 (Lateral force seismic method) for $a_{g,max}$=0.20·g (which corresponds to the input accelerogram), ground type B with $V_{s,30}$=456m/s, elastic response specta Type 1 (damping 5%) and importance factor γ_I=1.0. The surrounding soil was modeled using linear-elastic springs. Their stiffness k_h has been assessed according to the Vesić solution [16]. The piles, pile caps, beams, and columns were modelled using a beam finite element with appropriate material and geometric characteristics. This analysis was performed using the software Tower 6 (Radimpex). The results of this analysis, with the working title LE static seismic analysis – EC8, will be compared to the results of DRM analyses.

4.2 Model for numerical analysis

In order to perform the previously described dynamic analyses, the numerical model of the analysed SPS system is formed in the software Real-ESSI Simulator (Real-ESSI software), which was developed by Professor Jeremic from UC Davis, California (see Fig. 5). The software is based on the Finite element method. In order to form this model, except for all layers of the foundation soil, it was necessary to define two additional materials. One material for the DRM layer and the other material for the Damping layer. For both of these layers, all material characteristics are adopted as for layer 1 of the foundation soil, except material damping. Rayleigh damping is used in the dynamic analyses. The Rayleigh damping ratio ξ for the DRM layer is equal to zero (no damping), and for the Damping layer is 0.50.

It is well-known that the damping level of seismic waves in the soil increases with the increase in the level of plastic shear deformations in that soil caused by these waves. For this reason, higher values of the Rayleigh damping ratio ξ are adopted for the soil layers 1, 2 and 3. As expected, the value of this ratio was the highest for the softest layer. A standard value of damping ratio ξ=0.05 is adopted for the bedrock (layer 4).

The layered foundation soil is modeled with two indentical vertical ”screen” of elastic three-dimensional (3D) hexahedral finite elements with eight nodes (b=l=h=1m). Displacements of all nodes in the direction of the Y axis are prevented. The connection of finite elements at the contact between two different layers of foundation soil is ”direct” i.e., interface finite elements are not used.

The pile cap beam is modeled with two indentical vertical ”screen” of elastic 3D hexahedral finite elements with eight nodes (b/l/h=1.0/1.0/0.25m). Displacements of all nodes in the direction of the Y axis are prevented.

All elements of the RC frame as well as the piles are modeled with elastic beam (1D) finite elements with two nodes (l=1m). For these elements, appropriate geometric and material characteristics are defined in accordance with the adopted dimensions of their cross-sections and the characteristics of concrete as a material.

Linear zero-thickness interface finite elements with appropriate normal (axial) stiffness K_N and shear stiffness K_S are used to model the contact between the pile cap beam and soil, i.e., the contact between the piles and surrounding soil. These elements have unlimited axial compressive strength and constant axial stiffness, without tensile strength (stiffness), with constant shear stiffness until the shear strength τ_f is reached. After that, they are without shear stiffness. Of course, their shear strength depends on the level of normal stress, i.e., on the normal stiffness of K_N. Usually, this problem of the mutual dependence of normal and shear stiffness is solved iteratively. The adopted interface´s finite elements are without damping. The stiffnesses K_N and K_S are calculated using the following well-known empirical solutions:

$$K_N = \frac{E_{oed,i}}{t_i} = \frac{2 \cdot G_i \cdot (1-\nu_i)}{(1-2\cdot\nu_i)\cdot t_i} \tag{11a}$$

$$K_S = \frac{G_i}{t_i} = \frac{R \times G}{t_i} \tag{11b}$$

$$\tau_f = \sigma_n \cdot \mathrm{tg}(R \cdot \phi) \tag{11c}$$

where $E_{oed,i}$, ν_i, G_i and t_i denote the oedometric modulus of elasticity, Poisson's coefficient, shear modulus, and fictitious thickness of the interface finite element, respectively. In Eq. (11b) G denotes the shear modulus of the soil. The Poisson's coefficient of the interface finite element is usually 0.45, in order to avoid numerical errors that are common with these elements. The fictitious thickness of the zero-thickness interface finite element is usually from 0.01 to 0.1. In Eq. (11c), R denotes the strength reduction factor, which for the concrete-sand contact is from 0.8 to 1.0, while for the concrete-clay contact it is from 0.7-1.0. In Eq. (11c) ϕ denotes the angle of shear resistance of the soil.

4.3 Results

Figure 6 shows the deformed shape of the analysed SPS system at one moment of seismic excitation for the case 1×1C DRM. In the other images, the results of the performed dynamic analyses are shown in the form of recordings of the horizontal acceleration of the soil or structure during an earthquake. Also, horizontal acceleration elastic response spectra are shown. In those images, the black dashed line (Input) shows the input accelerogram used in the analyses. The black solid line (Output) shows the obtained recording of the horizontal acceleration or obtained horizontal acceleration elastic response spectra at the level of the bedrock for 1×1C DRM i.e. at the point with coordinates x_R=0, y_R=0 and z_R=-17m for 3C DRM. The blue solid line (Output_FF) shows the obtained recording of the horizontal acceleration or obtained horizontal acceleration elastic response at the central point on the soil surface of the model without RC frame and piles (free-field model), which was subsequently formed. The red solid line (Output_SPS) shows the obtained recording of the horizontal acceleration or obtained horizontal acceleration elastic response at the base of the structure, i.e., at the center point on the upper edge (z=0) of the pile cap beam.

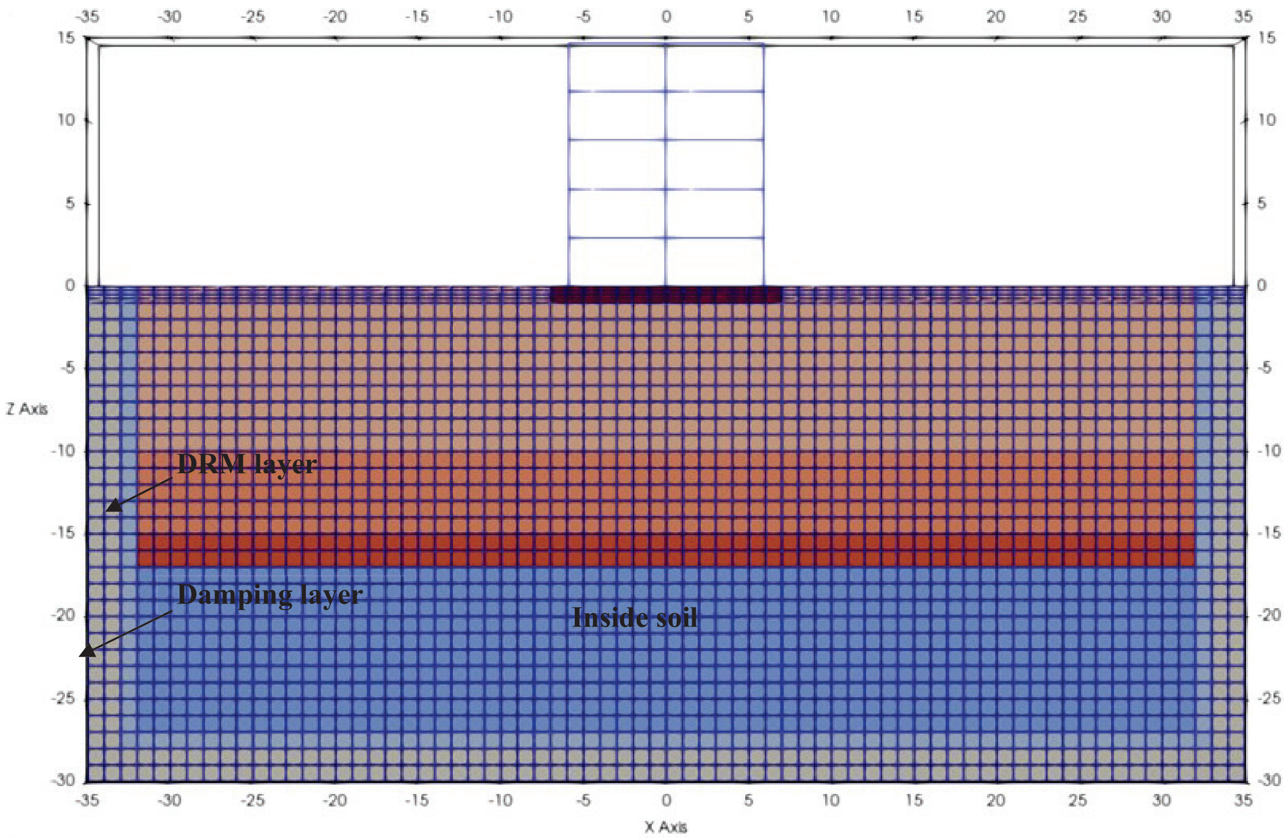

Figure 5. Numerical model of the analysed SPS system formed in Real-ESSI software

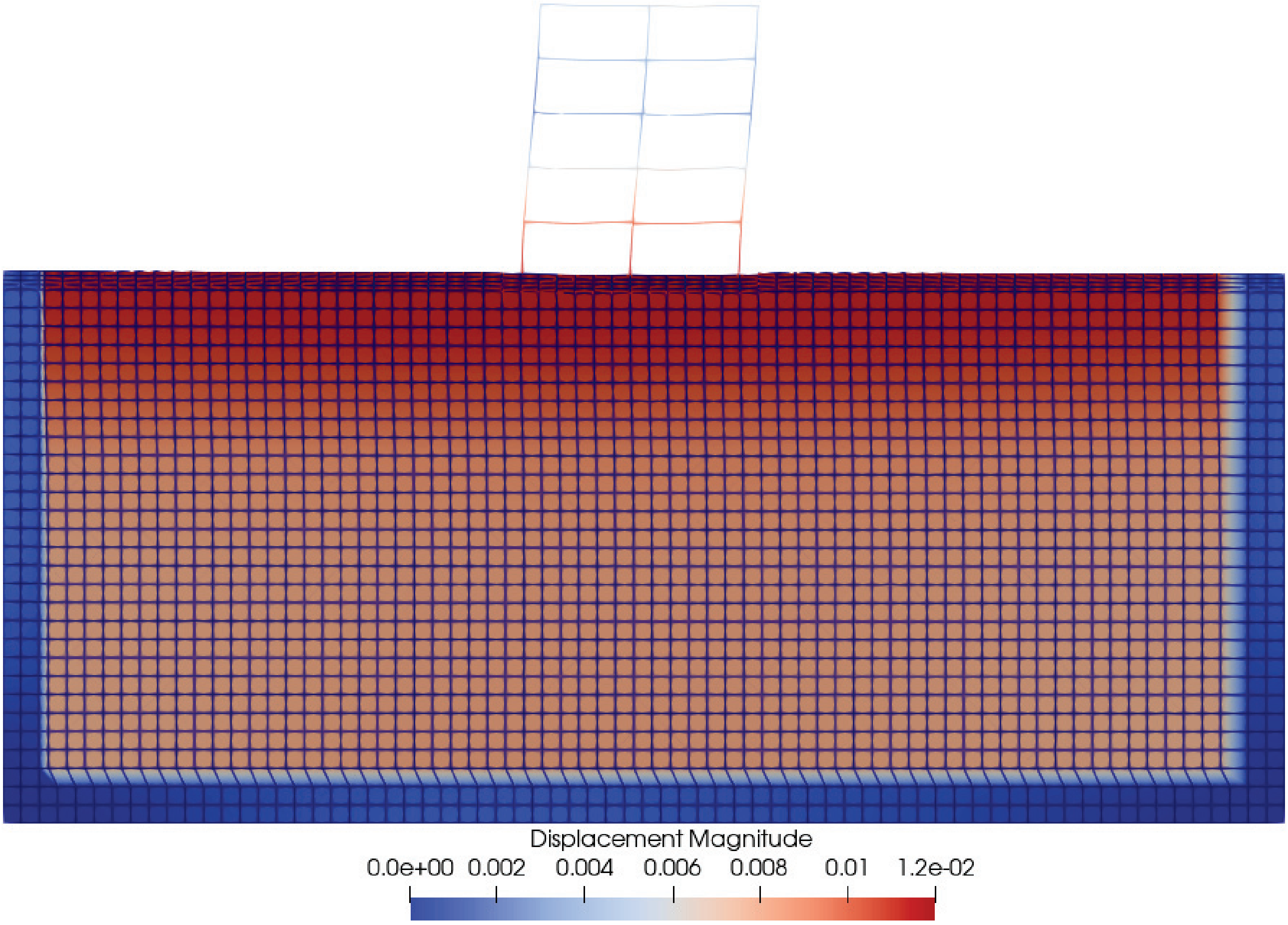

Figure 6. Deformed shape of analysed SPS system at the moment t=5.37s of applied seismic excitation – 1×1C DRM

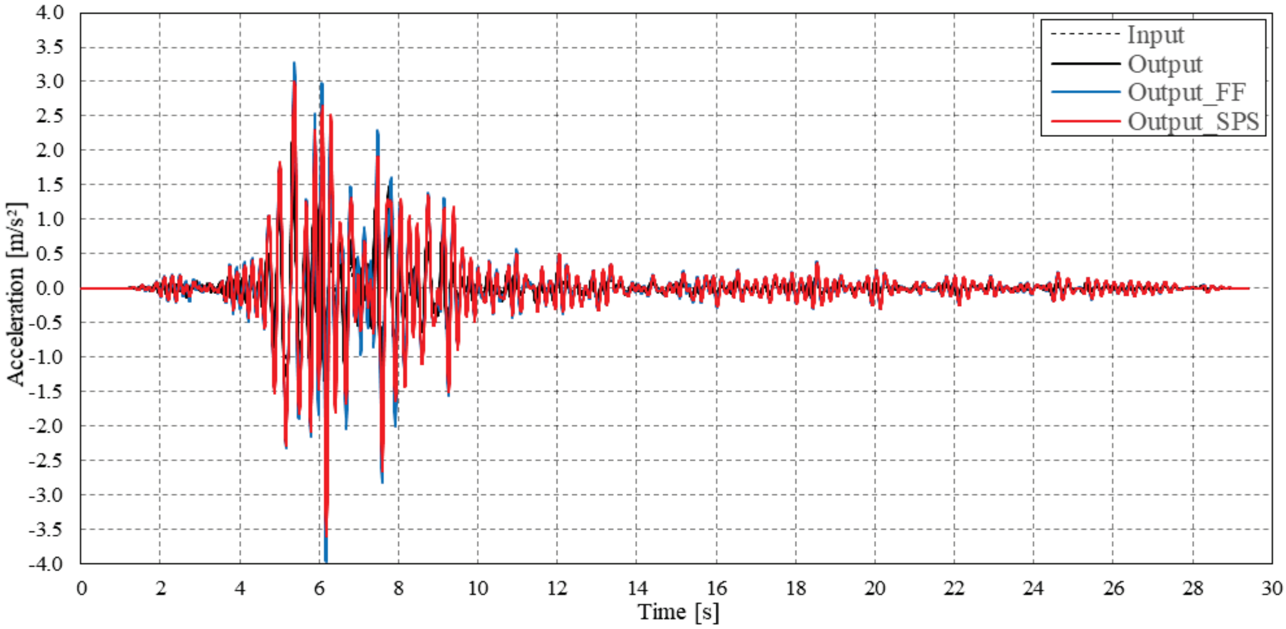

Figure 7. Horizontal acceleration recordings - complete time domain - 1×1C DRM

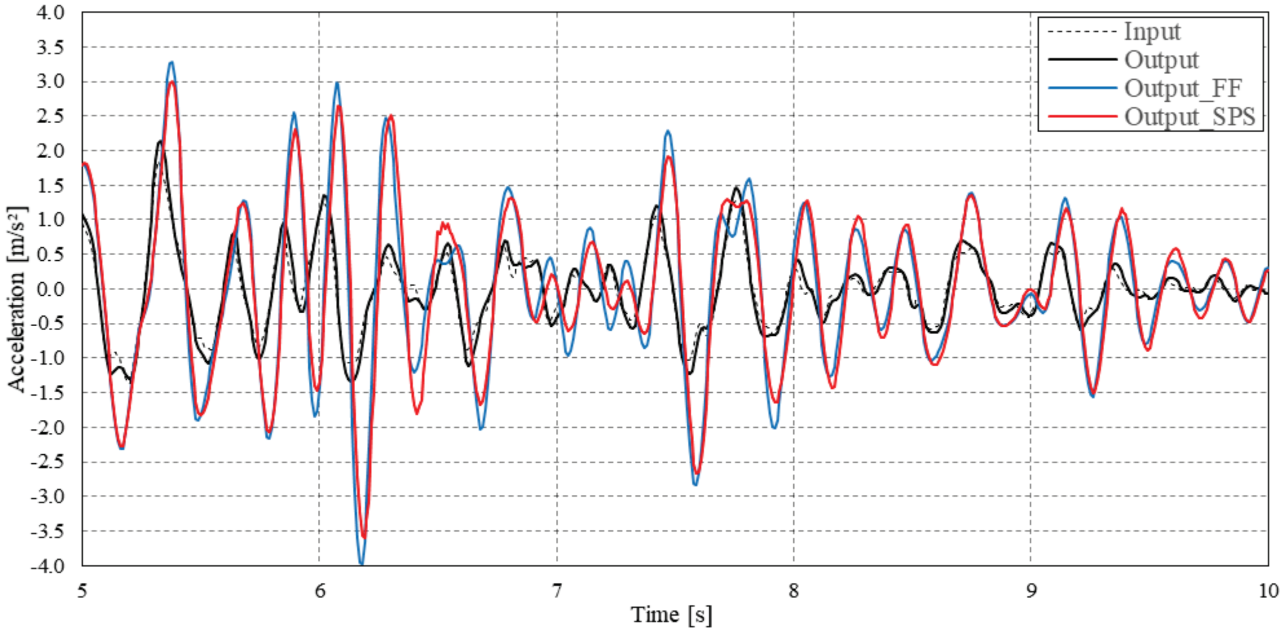

Figure 8. Horizontal acceleration recordings - interval between 5th and 10th second - 1×1C DRM

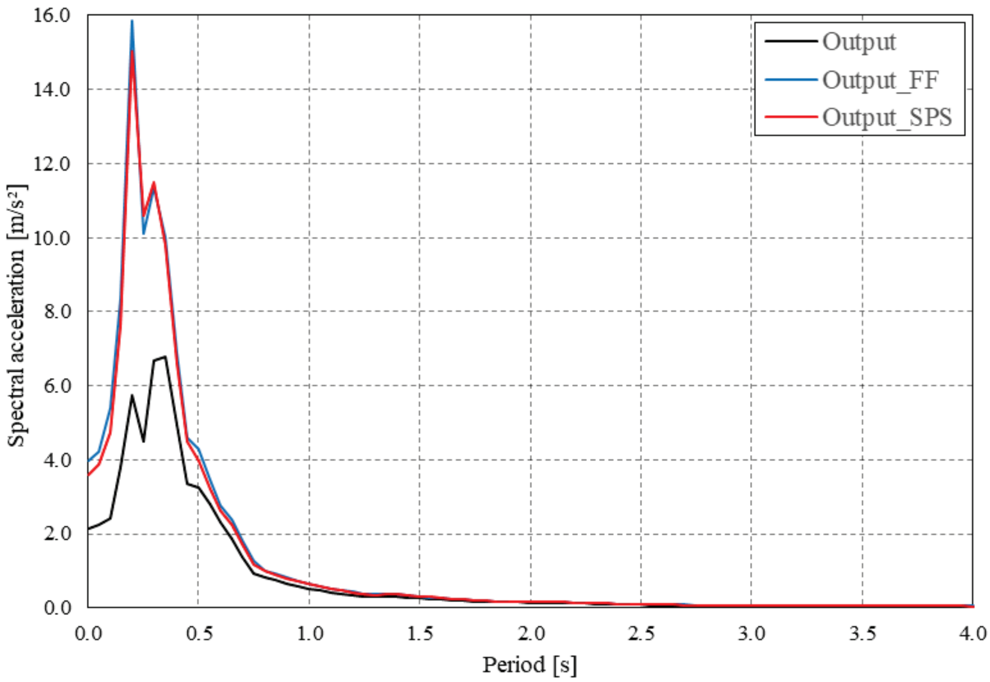

Figure 9. Horizontal acceleration elastic response spectra (ξ=5%) – 1×1C DRM

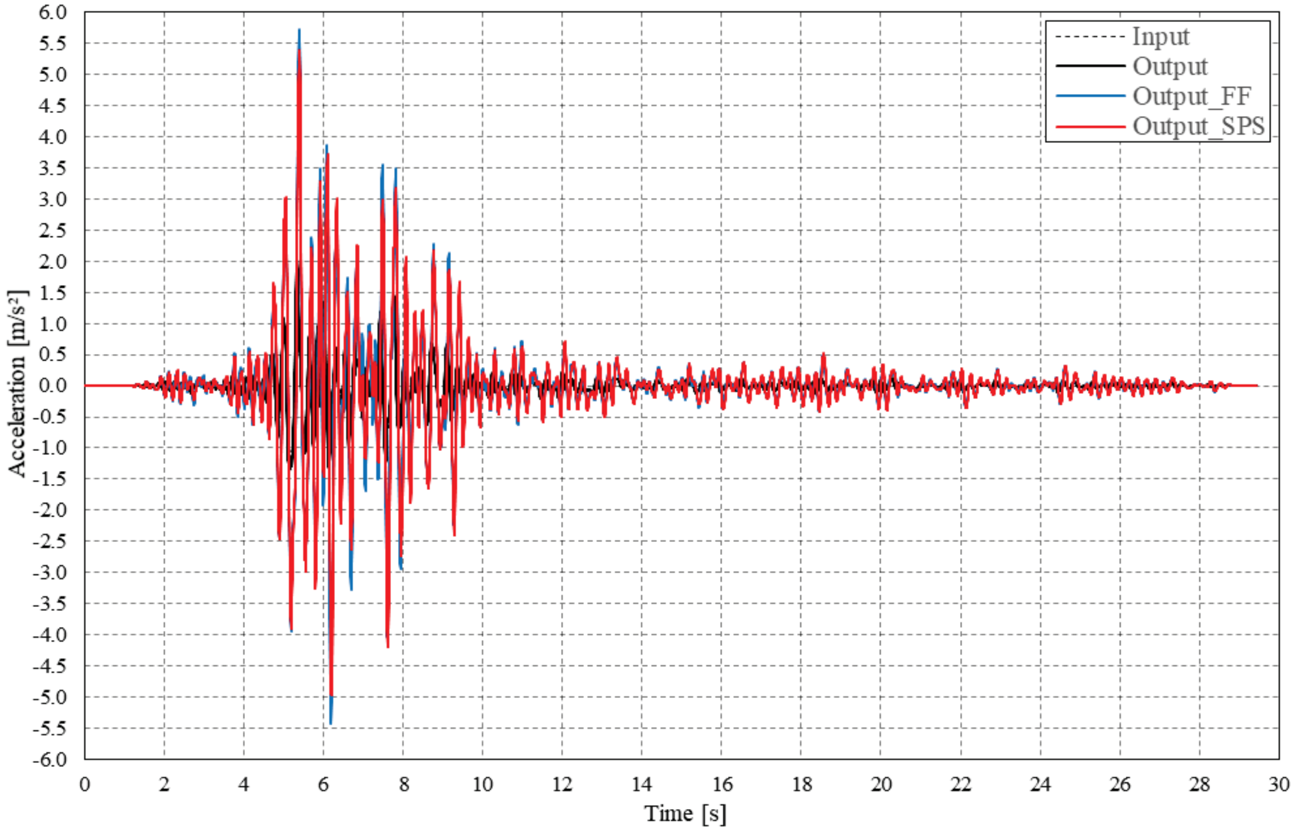

Figure 10. Horizontal acceleration recordings – complete time domain – 3C DRM

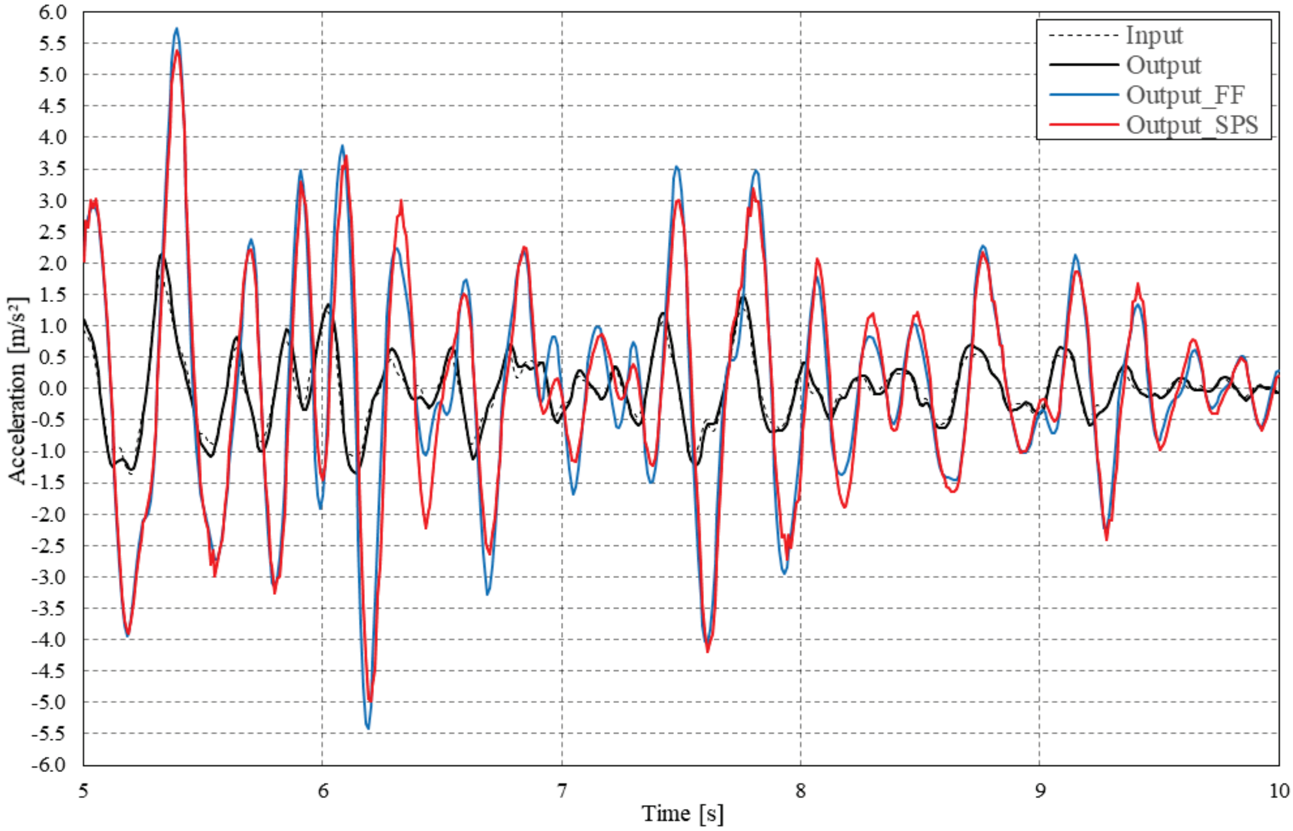

Figure 11. Horizontal acceleration recordings - interval between 5th and 10th second - 3C DRM

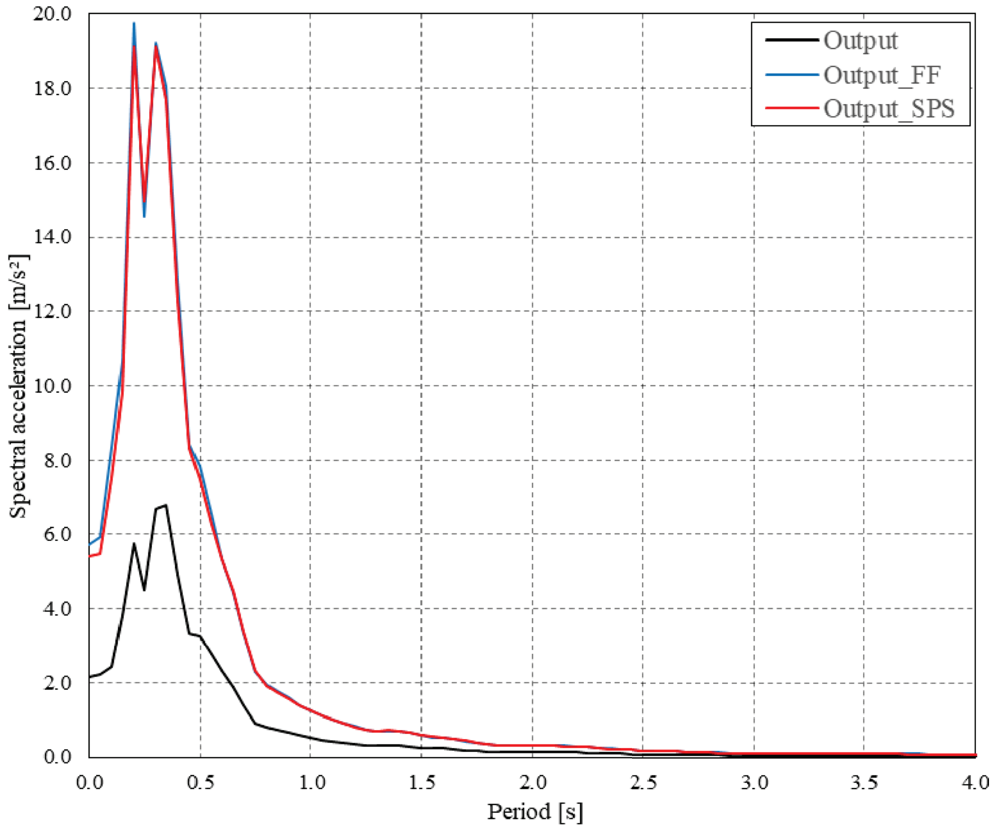

Figure 12. Horizontal acceleration elastic response spectra (ξ=5%) – 3C DRM

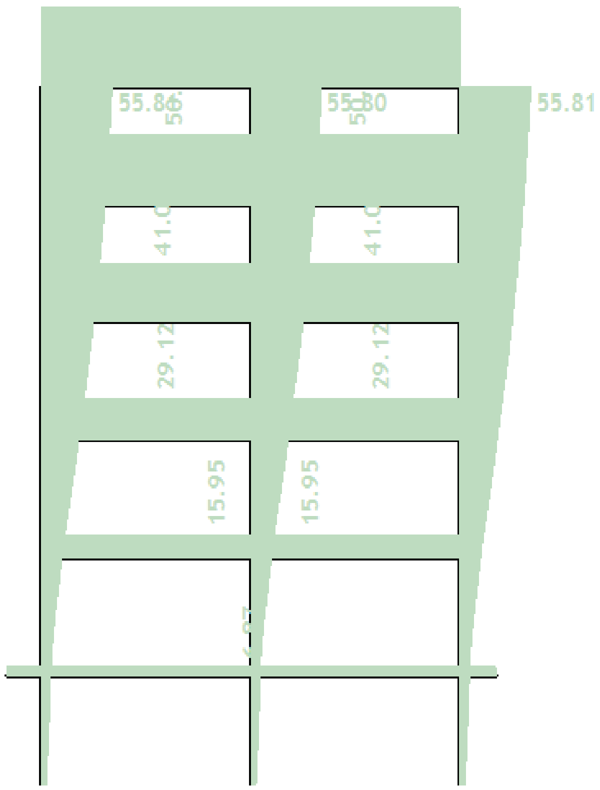

Figure 13. Horizontal displacement of RC pile-supported frame in mm – LE static seismic analysis – EC8 (Tower 6, Radimpex)

4.4 Discussion

Analyzing the presented results of the performed dynamic analyses, several interesting facts can be stated. Firstly, as a confirmation of the accuracy of the performed dynamic analyses, it can be stated that the horizontal acceleration recordings at the level of the badrock ''Input'' and ''Output'' match practically perfectly. Secondly, it can be stated that very similar horizontal acceleration recordings ''Output_FF'' and ''Output_SPS'' are obtained for both types of DRM. The same applies to their response spectra. So, in this case, the piles follow the displacement of the surrounding soil during the earthquake and almost do not affect the seismic excitation of the superstructure (RC frame), regardless of the fact that piles are wedged in the bedrock at their lower end. Of course, the question is what would happen in cases with a larger number of piles of the same or larger diameter, which will be the subject of some future research. Thirdly, it is very important to note that in the case of the 3C DRM, a significantly higher maximum horizontal acceleration of the structure at the level of the pile cap beam was obtained compared to this acceleration in the case of the 1×1C DRM. This is quite obvious if the corresponding accelerograms, i.e., the corresponding horizontal acceleration elastic response spectra, are compared. Therefore, in the case of the 3C DRM, the superstructure is exposed to stronger lateral seismic forces. For this reason, the horizontal displacement of the top of the RC frame, which in the case of the 3C DRM is 7.38cm, is almost twice as large as the horizontal displacement of the top of the RC frame obtained in the case of the 1×1C DRM. Fourth, the area where the horizontal spectral acceleration is strongly amplified at the level of the pile cap beam compared to the level of the bedrock is much smaller in the 1×1C DRM than in the 3C DRM. This fact can be very important for the correct assessment of the lateral seismic load of structures. As expected, the zone of pronounced amplification of the horizontal spectral acceleration is located around the first (fundamental) natural time period of foundation soil, which is 0.237s. Finally, in LE static seismic analyses – EC8, horizontal displacement of the top of the RC frame is 5.58cm. It is significantly higher than in the case of the 1×1C DRM. However, it is significantly less (about 32%) than in the case of the 3C DRM. Greater horizontal displacement implies greater horizontal seismic forces.

5 Conclusion

The Domain Reduction Method (DRM) presented in this paper is significantly different from other methods used for seismic soil-structure interaction analysis. By applying the presented method, many aspects of this interaction, which are usually neglected in other methods for justified reasons, can now be analyzed. One of those aspects is the influence of surface seismic waves on the seismic response of the structure. In other words, it is about the influence of the different directions of propagation of body seismic waves through the soil profile from the source to the structure on its seismic response. This was demonstrated by performing linear-elastic, dynamic analyses of the seismic interaction of the foundation soil and the pile-supported structure using the DRM. It is about a specific type of seismic soil-structure interaction that cannot be analyzed in a sufficiently high-quality way using the usual seismic methods. Among other things, the obtained results point to the eventual possibility that the Lateral force seismic method, which is recommended by the Eurocode 8 standard for regular structures and which is most often used in engineering practice, underestimates the level of the lateral seismic load of pile-supported structures. Of course, a firm conclusion can be drawn from the results of much more extensive and detailed research. This is a topic that the author will study in detail in the coming period.

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CENTAR ZA PUTEVE I GEOTEHNIKU

U okviru centra posluju odeljenja za geotehniku, nadzor i terenska ispitivanja, projektovanje saobraćajnica, laboratorija za puteve i geotehniku. Značajna aktivnost centra usmerena je ka terenskim i laboratorijskim geološko - geotehničkim istraživanjima i ispitivanjima terena za potrebe izrade projekno - tehničke dokumentacije, za različite faze i nivoe projektovanja objekata visokogradnje, niskogradnje, saobraćaja i hidrogradnje, kao i za potrebe prostornog planiranja i zaštite životne sredine. Stručni nadzor, kontrola kvaliteta tokom građenja, rekonstrukcije i sanacije objekata različite namene, izrada studija, ekspertiza, konsultantske usluge, kompletan konsalting u oblasti geotehničkog inženjeringa, neke su od delatnosti centra.

Ispitivanje šipova

- **SLT metoda (Static load test)**
- **DLT metoda (Dynamic load test)**
- **PDA metoda (Pile driving analysis)**
- **PIT (SIT) metoda (Pile (Sonic) integrity testing)**
- **CSL - Crosshole Sonic Logging**

ИМС IMS INSTITUT IMS AD BEOGRAD

- **Ispitivanje šipova**
    - **Geotehnička istraživanja i ispitivanja – in situ**
        - **Laboratorija za puteve i geotehniku**
            - **Projektovanje puteva i sanacija klizišta**
                - **Nadzor**

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Tondach

Krovni sistemi

Najlepši krov u komšiluku

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MATEST

YOUR PARTNER IN TESTING

iT TECH

INELAS ERECO d.o.o

MATEST "iT TECH" KONTROLNA JEDINICA

JEDNA TEHNOLOGIJA MNOGO REŠENJA

IT Touch Techlogy je Matestov najnoviji koncept koji ima za cilj da ponudi inovativna i user-friendly tehnologiju za kontrolu i upravljanje najmodernijom opremom u domenu testiranja građevinskih materijala

Ova tehnologija je srž Matestove kontrolne jedinice, software baziran na Windows platformi i touch screen sistem koji je modularan, fleksibilan i obavlja mnoge opcije

iT TECH pokriva

I INOVATIVNOST
I INTERNET KONEKCIJA
I INTERFEJS SA IKONICAMA
I INDUSTRIJALNA TEHNOLOGIJA

SISTEM JEDNOG RAZMIŠLJANJA JEDNOM SHVATIŠ - SVE TESTIRAŠ

NAPREDNA TEHNOLOGIJA ISPITIVANJA ASFALTA

I GYROTRONIC - Gyratory Compactor
I ARC - Electromechanical Asphalt Roller Compactor
I ASC - Asphalt Shear Box Compactor
I SMARTRACKER^{tm} - Multiwheels Hamburg Wheel Tracker, DRY + WET test environment
I SOFTMATIC - Automatic Digital Ring & Ball Apparatus
I Ductilometers with data acquisition system

MULTIFUNKCIONALNI RAMOVI ZA TESTIRANJE

I CBR/Marshall digital machines
I Universal multispeed load frames
I UNITRONIC 50kN or 200kN Universal multipurpose compression/flexural and tensile frames

OPREMA ZA GEOMEHANIČKO ISPITIVANJE

I EDOTRONIC - Automatic Consolidation Apparatus
I SHEARLAB - AUTOSHEARLAB - SHEARTRONIC Direct / Residual shear testing systems
I Triaxial Load Frame 50kN

MIXMATIC - Automatic Programmable Mortar Mixer

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