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POTRESNA VARNOST PREFABRICIRANIH KONZOLNIH ELEMENTOV ZA PREPREČEVANJE TOPLOTNIH MOSTOV

SEISMIC SAFETY OF THE PRECAST BALCONY CANTILEVER ELEMENTS FOR PREVENTION OF THERMAL BRIDGES

izvleček

V prispevku je analiziran potresni odziv armiranobetonskih (AB) balkonov, ki so v stavbo vpeti s prefabriciranimi detajli za preprečevanje toplotnih mostov. Slednji so bili razviti za prevzem vertikalne statične obtežbe na področjih z nizko seizmično aktivnostjo, njihov prenos na potresno bolj aktivna območja pa doslej v večji meri ni bil raziskan. Tako se lahko na našem trgu pojavijo tudi detajli, ki so z vidika potresnega inženirstva pomanjkljivi in lahko poslabšajo potresni odziv stavbe oz. njenega dela. V članku je prikazana potresna analiza izbranih tipskih konzol pri vertikalnih pospeških, ki nastopijo zaradi delovanja potresa, zaključke pa lahko posplošimo tudi na druge podobne rešitve vpetja balkonov. Rezultati potresnih analiz so pokazali: 1) toplotno izolirane konzole so bolj podajne od vpetih konzol, zaradi česar mora biti previs takih konstrukcij omejen na dolžino 300–350cm, 2) potresna obtežba lahko povzroči dvig konzole, kar pomeni, da natezne napetosti nastopijo tudi na spodnjem delu prereza in 3) prefabricirani elementi, ki se pojavljajo na trgu, nimajo ustrezne spodnje armature. Iz tega razloga je možnost poškodb teh elementov precej večja kot pri vpetih AB konzolah. Statistično izračunana verjetnost pojava dviga konzole na območju Ljubljane, ki bi pomenila večje poškodbe, je sicer razmeroma nizka (cca. 3% v 50-letni življenjski dobi). Kljub temu pa je to več od največje sprejemljive stopnje potresnega tveganja pred porušitvijo. Rešitev predstavljajo vpetostni detajli, pri katerih je v spodnjem delu konzole poskrbljeno za ustrezno natezno armaturo.

abstract

In the paper the seismic response of precast balcony cantilever structural elements for prevention of thermal bridges was analysed. This solution has been developed in seismic non-prone areas with the main purpose of eliminating a thermal bridge at the point where the balcony is fixed to the building. The solution has been specially made to withstand vertical static loading, not accounting for eventual vertical seismic loads in the case of transferring the solution to more active seismic zones. This paper deals with the seismic analysis of existent precast cantilever elements exposed to vertical accelerations and has proven that some elements in the case of lifting are not sufficient from the safety point of view. The results of the main research results obtained by numerous seismic analyses can be summarized as follows: 1) the insertion of a precast load-bearing thermal insulation element increases the flexibility of RC fixed base cantilevers and therefore limits their length to 300–400 cm, 2) vertical seismic loads can result in the cantilever uplift, which means that tensile stresses could appear also at the bottom of the cross-section 3) precast elements, that appear on the Slovenian market to this day, do not have the appropriate steel reinforcement in the bottom part of the cross-section. For this reason, the possibility of damage is considerably higher for precast cantilever structural elements than for RC fixed base cantilevers. Statistically calculated probability of cantilever uplift for Ljubljana, which would result in severe damage, is relatively low (3% in the 50-year life span). However, the calculated value is greater than the maximum acceptable level of seismic risk for collapse. One of the possible solutions to prevent the negative influences of cantilever uplift is to consider the proper reinforcement also at the bottom of the precast elements' cross-section, or by other measures preventing uplift.

ključne besede

energijsko učinkovite stavbe, preprečevanje toplotnih mostov, potresna varnost, konzolne konstrukcije, prefabricirani elementi, vertikalna potresna obtežba

key words

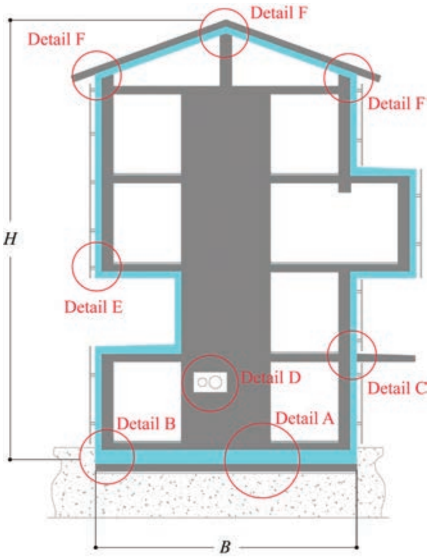
energy-efficient buildings, thermal bridges prevention, seismic safety, cantilever structures, precast elements, vertical seismic load

Introduction

The demand of constructing buildings without thermal bridges is a trend, which applies to all new built buildings, regardless of the different definitions of low-energy buildings and the use of different passive and active systems to reduce energy consumption. Already a small thermal bridge can endanger the environmental concept of such buildings [Feist, 2007]. The problem exposed in this article relates to the fact that the construction of low-energy buildings is also present in earthquake-prone areas. However, the specific details to prevent thermal bridges have not been adequately verified on dynamic seismic loads [Kilar et al., 2013]. Structural control for seismic load is necessary, because the majority of problematic junctions is resolved by inserting thermal insulative parts between the load-bearing structural elements and can cause weakening of the structure in the most crucial parts of the building. On the account of improving thermal comfort of the building structural integrity/stability can be threatened.

First low-energy buildings were low-rise buildings, which are not so vulnerable to the changes on the building envelope from the point of view of structural resistance [Zbašnik-Senegačnik, 2011]. The latter is the main reason, that structural seismic safety of low-energy buildings has not been thoroughly researched

A. Temeljenje na sloju toplotne izolacije.
B. Posebni izolacijski podstavki za preprečevanje toplotnih mostov sten, ki nalegajo na hladne elemente.
C. Detajli za preprečevanje toplotnih mostov pri konzolnih konstrukcijah (obravnavani v tem članku).
D. Prekinitev nosilne konstrukcije zaradi zahtevanega sistema kontroliranega prezračevanja.
E. Pritrjevanje fasadnih elementov.
F. Pritrjevanje strešne konstrukcije in zagotovitev toge strešne diafragme.



Slika 1: Shematska predstavitev detajlov energijsko učinkovitih stavb kritičnih z vidika njihove potresne odpornosti.
Figure 1: Figure 1: Schematic representation of low-energy buildings' details critical from the point of view of earthquake resistance.

until now. Solutions for new critical details in passive and low-energy buildings are mainly developed and experimentally tested by manufacturers of construction products and architecture designers according to the requirements of an individual building project. The special details of passive and low-energy buildings, which could be critical in the case of dynamic seismic loads, are shown in Figure 1 and can be divided as following:

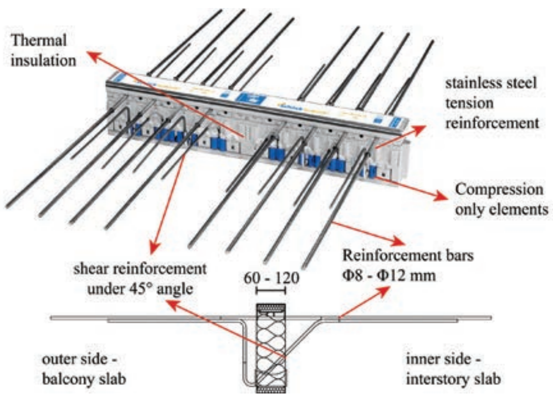
A. Installation of thermal insulation (TI) with suitable compressive strength beneath the ground floor slab, foundation slab or strip foundations,
B. Interruption of the thermal bridge at the junction of the outside wall with the strip foundation or foundation slab by means of a so-called insulation base made of a material with suitable compressive strength and thermal conductivity,
C. Special innovative solutions of different load bearing TI elements proposed by manufacturers of construction products for prevention of balcony cantilever thermal bridges (analysed in this paper),
D. Interruptions in the structural system, because of the new requirements of the controlled mechanical ventilation system,
E. The mounting of external façade elements and
F. Roof structure and ensuring the stiff roof diaphragm.

In the paper, precast load-bearing TI elements for prevention of balcony cantilever thermal bridges have been analysed (Figure 1, detail C). The models considered in the paper are regular, straight RC cantilevers with constant cross-sections and without additional supports, such as beams, cables etc. To this date, there is no relevant literature investigating the seismic response of such elements, although they are one of the basic components of energy-efficient buildings, which effectively prevent thermal bridges for balcony cantilevers. In the case of older buildings, where thermal bridges have not yet been adequately addressed, the surface temperature on the junction of the balcony cantilever and the external wall reduces to the condensation point, which causes constant humidity and mould problems. In addition, the impact of cantilever thermal bridges increases the use of energy for heating, which cannot be ignored in modern energy-efficient buildings. Furthermore, from the architectural design perspective the desire to extend the length of balconies and to achieve diverse architectural design (Figure 2) also raises the question of structural safety for such thermally insulated cantilevers. At this point we presume that the seismic response of cantilevers with load-bearing TI elements is more critical compared to the regular RC cantilevers.

Slika 2: Primer stavb z razvejanimi konzolnimi konstrukcijskimi elementi: projekt Panzerkreuzer Wohnanlage na Dunaju s prefabriciranimi toplotnoizolacijskimi elementi. [Rüdiger Lainer + Partner, 2008]
Figure 2: Example of buildings with cantilever structure elements: project Panzerkreuzer Wohnanlage in Vienna with load bearing thermal insulation elements. [Rüdiger Lainer + Partner, 2008]

General application of precast cantilever elements for prevention of thermal bridges

In regular cantilever structures with no TI (e.g. RC and steel cantilever beams or slabs), relatively high heat losses are present, as well as significant decrease in surface temperatures. The latter leads to higher heating costs and unhealthy mould on the inner side of the detail [Goulouti et al., 2014, Ge et al., 2013]. Ge and co-workers analysed the effects of balcony thermal bridges on the use of energy for heating in a 26-floor building. They have concluded that the area of these thermal bridges covers 4% of the total building area, and thus the energy use for heating is increased by 5–11% on an annual basis for a given building.
The solution proposed by most of the precast elements manufacturers [Schöck, 2014, Max Frank, 2013, H-BAU, 2014, HALFEN, 2014] is designed to disconnect the load-bearing structure, which exposes high thermal conductivity, and replace it with thermal insulation. The selection of precast element is dependent on the structural system, material of the structure, cantilever length and the amount of expected serviceability load. Most widely used TI in such precast elements is expanded polystyrene (EPS) with thickness between 6–12 cm. The proposed solution is extremely effective from the thermal performance point of view. However, it is more vulnerable to structural failure, since the weakening of the structure is positioned exactly at the cross-section with highest internal



Slika 3: Prefabriciran nosilni toplotnoizolacijski element Schőck Isokorb tip A-K [Schöck, 2012] in njegov vzdolžni prerez.
Figure 3: Prefabricated load bearing thermal insulation element Schőck Isokorb tip A-K [Schöck, 2012] and its cross section.

forces. The latter can be illustrated by a strength comparison of EPS, whose nominal compressive strength is usually lower than 200 kPa, and concrete C30/37, which exposes up to 100 times greater strength. Due to such strength difference, the manufacturers of precast elements additionally strengthen these elements by adding compressive bearings, which are placed at the bottom of the precast cross-section [Schöck, 2014, Keller et al., 2007]. These compressive bearings in most cases consist of micro-steel fibre reinforced high-performance concrete and also of synthetic polymers reinforced with fiberglass. In some cases of precast elements with higher bending strength the compressive bearings are made of stainless steel studs. The transfer of forces due to the negative internal moment is achieved by tensile reinforcement on upper side of the cross-section and by compressive bearings at the bottom. In the paper, an agreement that negative moment causes tensile stress on the top edge of the cross-section is considered. The details are usually consisted of stainless steel reinforcement, which is according to the manufacturers used due to its smaller heat transfer coefficient, so that the detail is improved in terms of energy efficiency. Given that the cross-section of longitudinal reinforcement is much smaller compared to the total precast element cross-section, we can conclude, that this contribution is negligible. In addition to compressive bearings, which also provide some shear resistance, reinforcing bars inclined at an angle of 45° are used for the transmission of shear loads.

Analysed examples

In the first part of the paper, the response of thermally insulated RC cantilevers (TIC model) on vertical static and seismic loading is presented. It is important to note that only vertical oscillation of the cantilever was analysed. In this part of the paper also a comparison is shown with the fixed based RC cantilevers (FBC models), which were selected with the same dimensions (height of the cross-section, amount of steel reinforcement and concrete grade). Selection process was based on the design of TIC models on vertical static loads, for which we relied on the precast elements catalogues [Schöck, 2014, Max Frank, 2013, HALFEN, 2014, H-BAU, 2014]. As it turned out the FBC models with the same characteristics (the same amount of tensile reinforcements and geometric data) expose higher bending strength capacity than the corresponding TIC models, so the design according to TIC models is sufficient. For the purpose of the study 5 models with different lengths (Table 1) were analysed, where the longest cantilevers (400 and 500 cm) consisted of load-bearing TI elements with the strongest capacity offered by manufacturers in their catalogues. Special cantilevers with extreme lengths are, however, usually designed individually in collaboration with architects and structural engineers (self-weight reduction, additional steel beams, additional supports, etc.). However, in this article such individual approach was neglected, since we wanted to discover the response of TIC models in seismic-prone areas.

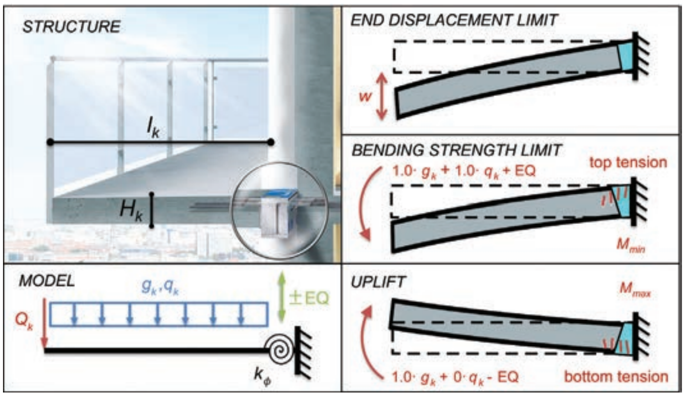
The description of the TIC model together with analysed loads and corresponding labels is presented in Figure 4. The weight of the RC slab (25 $^{kN}/_{m3}$ •H_k[m]) and concrete screed (1.5 kN/m^2) was considered for the calculation of self-weight and dead loads. Serviceability load was added as a combination of one point load at the end of the cantilever (Q_k= 1.0 kN) and linear uniformly distributed load (q_k= 5.0 kN/m^2) [CEN, 2004]. For seismic design combination different proportions of serviceability loads were taken into account (0, 15, 30 and 100 %). The latter is necessary to include all possible events during earthquakes, such as unused balconies, fully functional balconies or intermediate combinations. The RC slab was modelled with a 2D line element and effective width 1 m (cantilever dimensions: 1 m/H_k/l_k). The cross-sections of both models (FBC and TIC) were analysed as uncracked RC sections, while the difference between the models is mainly in the boundary conditions. A fully rigid support was considered for all FBC models, whereas for TIC models a rotational spring including the flexibility of the load-bearing TI element was added. The stiffness of the rotational spring is dependent on the height and type of the load-bearing TI element (Table 1).

Next to the labels and model presentation, also the possible critical failure states induced by seismic loads are presented in Figure 4. As the first limit case, the exceedance of the maximum allowed end deflection was recognized. Since the load-bearing TI elements are in fact extremely flexible, it is in some cases possible to reach large deflections even before the structure suffers severe inelastic damage. The limit value for the critical end deflection can be quite different, depending on whether it is necessary to protect the (non-) structural elements or not (e.g. winter garden, glass fence, etc.). Critical deflections of the cantilever can be prevented if the building designer and investor decide there is too much risk for excessive damage of secondary (non-) structural elements. The second possible failure mechanism occurs, when the maximum bending strength of the

Cantilever length (l_k) [cm]	100	200	300	400	500
Cantilever height (H_k) [cm]	16	20	22	25	25
Tension reinforcement [mm]	8 ϕ 8	16 ϕ 8	11 ϕ 14	12 ϕ 14	12 ϕ 14
Shear reinforcement [mm]	4 ϕ 6	4 ϕ 8	8 ϕ 8	8 ϕ 8	8 ϕ 8
Moment strength (M_{Rd}) [kNm]	-14.8	-43.1	-94.2	-126.9	-126.9
Rotational stiffness (k_Φ)* [kNm/rad]	1465	6099	9045	10050	10050

*Equation for rotational stiffness is taken according [Schöck, 2014].

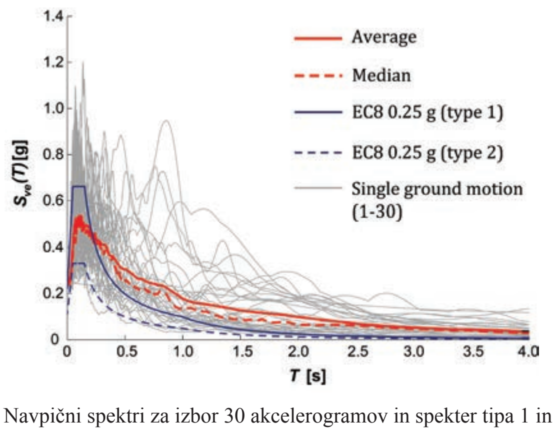
Tabela 1: Vhodni podatki za analizo izbranih nosilnih toplotnoizolacijskih elementov.
Table 1: Input analysis data for load-bearing thermal insulation elements.



Slika 4: Shematska predstavitev TIC modela s pripadajočimi oznakami ter prikaz možnih kritičnih stanj.
Figure 4: TIC model representation with corresponding labels and possible critical states.

load-bearing TI element is exceeded. This failure mechanism can be recognized by visible cracks and damage on the top side of the RC slab or load-bearing TI element. Such critical state is more likely to occur, if the balcony is fully functional (the maximum amount of serviceability load is applied). For such failure, the tensile strength in the upper part (tensile stress in steel reinforcement) or the compressive strength in the bottom part of the cross-section (compressive bearings for TIC models or RC slab section for FBC models) is exceeded. The latter is possible only for extreme seismic loads, since the cantilevers are already designed with large safety factors for vertical static loads. For the last critical state the phenomenon of cantilever uplift is shown. Uplift occurs if the seismic excitation in the opposite gravity direction is stronger than vertical static loads (self-weight and dead loads). The occurrence of such border mechanism is therefore more likely in the case of unused balconies (no serviceability loads) exposed to vertical seismic excitation. On account of cantilever uplift the stress diagram in the cross-section significantly changes, so that even the tensile stresses are possible in the lower part of the section. This situation will not be critical if the cantilever exposes sufficient tensile strength also in the lower part of the section. Otherwise, stronger damage on the lower part of the cantilever cross-section is highly likely.

In the paper, linear elastic analysis with vertical response spectrum from EC8 was performed [CEN, 2005]. In addition, at the end of the paper some results from the seismic risk assessment (non-linear incremental dynamic analysis) were added. The seismic loads for both analyses are shown in Figure 5. As it can be seen from Figure 5, the EC8 standard allows two different vertical response spectrums, where type 2 spectrum is used only for earthquakes of magnitude not greater than 5.5. Seismic excitations in vertical and horizontal direction are significantly different, which is reflected also in different shapes of vertical and horizontal response spectrum. In our analysis, type 1 vertical response spectrum was considered. This response spectrum is used for all soil types (A-E) and its acceleration is 10% lower than the horizontal spectrum acceleration (a_{vg}/a_g =0.9). The results of the analysis are presented for various seismic intensities (a_g=0.15–0.45 g), where an agreement for presenting the acceleration in the scale of corresponding horizontal seismic intensities is taken into consideration. In Figure 5 the selected 30 ground motion records, which have been considered for probability analysis, are shown as well. From the Figure 5 it is evident that the analysed 30 ground motions' median/mean values are within the range of EC8 vertical response spectrum type 1 and 2. The latter is somewhat expected, since the response spectra given in the EC8 standard are conservative and calculated as an envelope of realistic earthquakes, so they can be used in design with no limitations.

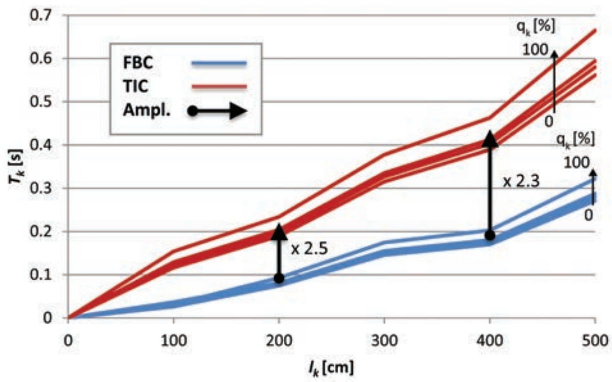


Slika 5: Navpični spektri za izbor 30 akcelerogramov in spekter tipa 1 in 2 iz EC8 pri izbrani intenziteti 0.25 g.
Figure 5: EC8 vertical response spectrum (type 1 and 2) and median response spectrum for 30 ground motions for a selected seismic intensity of 0.25 g.

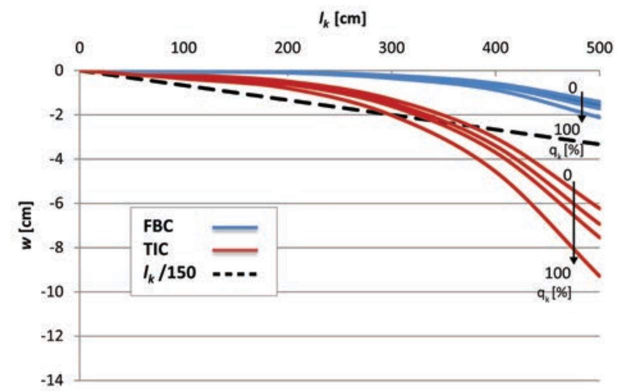
Analysis results

Vertical static loads

In the first part, the results for vertical static load analysis are given. In this part we wanted to point out the results for already existing load-bearing TI elements, which originate from seismic non-prone areas. Some of the analysed models are, however, in practice not possible, since the load-bearing TI elements do not reach sufficient carrying capacity or their end deflection reaches extremely high values (usability of such elements is impaired). As mentioned in previous sections, the emphasis in design of the TIC model is given for end deflection (w), which increases considerably compared to the FBC model's deflection. Cantilever end deflection is in the case of TIC models dependent on the concrete slab deformation and rotational deformation of



Slika 6: Nihajni čas FBC in TIC modelov (T_k) v odvisnosti od deleža koristne obtežbe.
Figure 6: Fundamental period of vibration for FBC and TIC models (T_k) dependant on the variable load.



Slika 7: Poves konca konzole v mejnem stanju uporabnosti v odvisnosti od dolžine konzole in velikosti koristne obtežbe.
Figure 7: Cantilever end deflection for the serviceability load combination dependant on the variable load and cantilever length.

the load-bearing TI element. A more clear graphic demonstration of increased TIC models' rotational flexibility can be obtained from Figure 6, where the fundamental period of vibration is shown as a function of serviceability load and cantilever length. Fundamental period of vibration is the main parameter for determination of seismic forces (Figure 5), in particular, it shows information about the rigidity of the models. From the Figure 6 it can be determined that the fundamental period of vibration for TIC models is in all cases greater than the corresponding FBC model period. Approximate factor of increase, which can be expected regardless of length or serviceability load, is between 2.0 and 2.5. In general, this means that TIC models are more flexible and will oscillate up to 2.5 times slower than FBC models. Additionally, in Figures 6–8 different curves within the same colour determine the results for different scale of serviceability loads. It can be concluded from Figure 6, that the scale of the serviceability load has a smaller impact on the period increase than the difference in models (FBC and TIC). For a given example of TIC model with length 300 cm, the difference in the fundamental period of vibration between the model with no serviceability loads and fully loaded model equals up to 20%. The increase in the fundamental period of vibration is also reflected in larger values of TIC models' end deflections (w). In Figure 7, end deflection is presented for all of the analysed models (FBC and TIC) subjected to vertical static loads. If we compare the results with the serviceability limit state (w_max=l_k/150), which is defined in the EC0 [CEN, 2004], than it can be concluded, that all of the FBC models are sufficient (regardless of their length). The serviceability limit state l_k/150 is to a certain extent a strict requirement, however, it prevents visual impairment and general operability of the structure. In most cases, the cantilever will not be damaged in the case of reaching the serviceability limit state and will return into its original position after reloading. Larger values of end deflections are present for TIC models, where the differences with comparable FBC models equal even to a factor of 5 and more. If the limit value from serviceability limit state (l_k/150) is compared with results from Figure 7, the maximum length of the cantilever, for which the application of load-bearing TI element is still possible can be determined. As we can see, the TIC models are critical for lengths above 300 cm. In cases with lower serviceability loads, even longer cantilevers with load-bearing TI elements are possible (the limit is reached at 380 cm). The main limitation of load-bearing TI elements is therefore not in their small carrying capacity, but in the large flexibility, which can result in exceeded end deflection even for vertical static loading. In this case, it can be concluded, that the load-bearing TI elements, which appear on Slovenian market, can be used for cantilevers shorter than 300 cm.

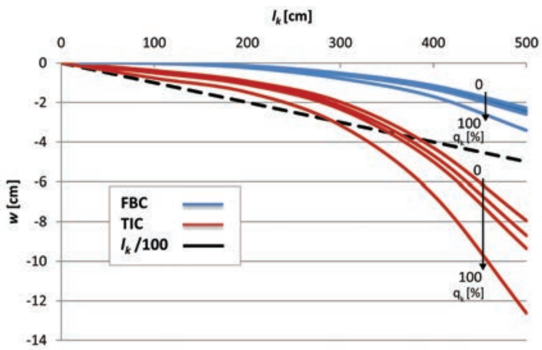
Vertical seismic load

In the event of earthquakes, we can expect even greater end deflections as we have specified for vertical static loads, however, it is necessary to point out that earthquake load is only temporary. In such extreme cases even greater end deflections are allowed. The EC8 standard does not regulate maximum end deflections in the event of earthquakes, so the decision to limit these deflections is left for architects and building engineers. In addition to protect the primary load-bearing structure (RC slab for FBC models and load-bearing TI element for TIC models) it is also sensible to protect certain secondary (non-)structural elements, which could be highly damaged during earthquakes. In Figure 8 end deflections are calculated by response spectrum analysis for different scales of serviceability loads and for seismic intensity of 0.25 g. For the critical value of end deflections, a higher value than for serviceability limit state is presumed. Computational limit of l_k/100, which is marked with a black dashed line, is only an indicator, which provides a relation to large deflections. If this computational limit was considered, the maximum allowed length for the use of load-bearing TI elements would be 380 cm for models with no serviceability loads and 300 cm for models with full serviceability loads applied. As we can see, the limit lengths are in this case approximately the same than for serviceability limit state.

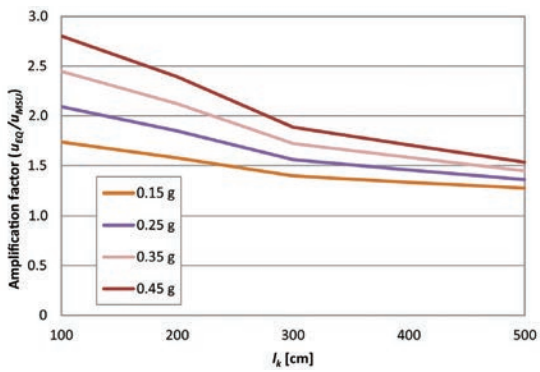
For larger seismic intensities (a_g>0.25 g) the end deflection increases, as shown in Figure 9. Factors of increase are calculated according to the serviceability design combination deflections (Figure 7). The results are shown for models with the same scale of serviceability load applied (fully loaded cantilever). From the Figure 9 it can be concluded that next to the seismic intensity, factor of increase depends on the length of the cantilever as well. The increase is greater for shorter cantilevers, which means that the seismic load has a greater impact on short, stiff cantilevers. This conclusion can also be drawn from the shape of the vertical response spectrum (Figure 5). The factor of increase rises with the scale of seismic intensity and it reaches values between 1.5 and 3.0 for short cantilevers, depending on the seismic intensity (for models with l_k=100 cm the factor of increase equals to 1.75 and 2.8 for the seismic intensity 0.15 g and 0.45 g, respectively). In Figure 10, the maximum internal moment at the cantilever fixed end is presented. The results are divided by negative moment resistance for each of the individual models (M_{Rd}) and are shown as a function of cantilever length and different seismic intensity. Cantilever uplift in the event of earthquakes is defined by the occurrence of positive internal moment at its fixed end. Uplift is therefore possible if earthquake load acts in the opposite direction of gravity and decreases vertical static loads effect (self-weight and dead loads). The bar graphs, which are above the horizontal dashed line, indicate positive internal moment and uplift respectively. From the results in Figure 10, it can be concluded that the uplift phenomenon is more critical in the case of short cantilevers, since positive internal moment appeared only for models shorter than 200 cm. For cantilevers shorter than 100 cm, uplift appears already for seismic intensity of 0.25 g, while for longer models higher seismic intensities are necessary. The seismic risk of cantilever damage due to the occurrence of uplift is greater for TIC models, since they do not provide tension reinforcement in the bottom part of the cross-section. This means that in the case of tensile stresses at the bottom of the cross-section, the compressive bearings (Figure 3), which expose negligible tensile strength, will be damaged. To avoid such damage, additional safety measures are necessary (e.g. the selection of improved load-bearing TI precast elements). The solution to reduce the seismic risk of TIC models' uplift is to consider the EC8 provisions for local ductility of RC cross-sections. Some manufacturers already offer precast elements with tensile reinforcement on both ends, which could meet the requirement of the EC8 standard. However, these elements are designed primarily for continuous RC slabs and are not intended for free cantilevered balconies in seismic-prone areas. A development of load-bearing TI elements should in the future be continued also in the direction of improving their response on seismic loads and thus reduce their potential damage. Some attempts to include additional tension reinforcement at the bottom of the precast elements can already be found in the latest catalogues of manufacturer Schöck. Such seismic elements at least partially solve the problem of uplift. However, the results of our analysis show that the amount of tension reinforcement in these elements (2 ϕ 8 mm) is not sufficient in the case of moderate and strong earthquakes. At this point it is necessary to point out the importance of building designers (architects, civil engineers etc.), who must be aware that some of the already developed details cannot be transferred to seismic-prone areas.

To further investigate the changes in seismic response between the FBC and TIC model, we performed the seismic risk assessment, similar as in [Dolšek, 2012]. Case study examples of TIC and FBC models in length 300 cm were analysed with non-linear dynamic analysis and the probability of various limit states (Figure 4) was assessed. Such probability assessment is used for performance based seismic design and is beneficial from the perspective of building investors and designers, which can decide what is their acceptable level of risk. For example, in the case of extremely high probability of cantilever damage in its intended life span, it is sensible to enhance the structure detail and decrease such risk. This would improve the seismic response and reduce the repair costs respectively. With such approach you can predict the response of the structure in its entire lifetime already in the design stage, with more precise models. In the paper, two different nonlinear models (FBC and TIC) were considered. For TIC model, negligible positive internal moment strength is considered, as these elements do not have tension reinforcement at the bottom of the cross-section. On the contrary, the FBC model was reinforced also at the bottom of the section. With this measure, the negative consequences due to the cantilever uplift are effectively prevented. For seismic risk assessment, the only source of uncertainty was the dispersion in the records of actual earthquakes, which were chosen on the basis of a statistical sample. The results amplifications due to the record variability were somewhere between 1.6–2.9 for FBC models and 2.6–3.9 for TIC models. High amplification factors are the consequence of high variability of 30 records considered (Figure 5). It can be concluded from the given amplification factors that TIC models are more susceptible for increase in the results scatter. This is due to the property of the TIC model, which is unable to prevent uplift and responds extremely asymmetrically (i.e. only some of the 30 earthquakes will cause positive internal moments). The seismic hazard function for Ljubljana was defined according to the design ground acceleration on firm soil in EC8 and their corresponding return periods [Lapajne and Šket Motnikar, 2001].

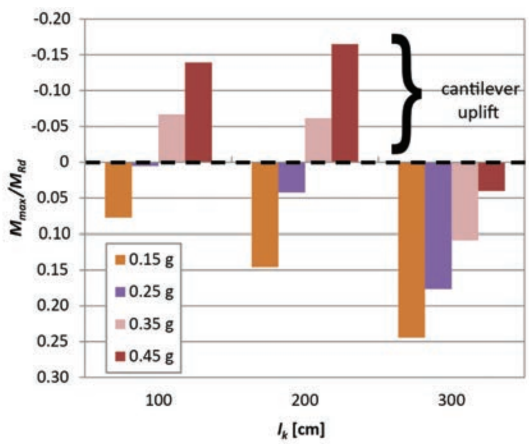
In Table 2, the results of the median annual frequency for different limit states (λ_{LS}) taking into account the amplification factors due to the record variability are given. Furthermore, the calculated median frequencies are presented also for the assumed lifetime (50 years) of the structure (λ_{LS}^{50}). The seismic risk assessment for both models has confirmed the assumption that TIC models' seismic response is impaired if compared to FBC models. The probability of severe structural damage or



Slika 8: Poves konca konzole v potresni obtežni kombinaciji za potresno intenziteto 0.25 g v odvisnosti od dolžine konzole in velikosti koristne obtežbe.
Figure 8: Cantilever end deflection for the earthquake load combination of seismic intensity 0.25 g dependant on the variable load and cantilever length.



Slika 9: Faktor povečanja povesa zaradi vertikalne potresne obtežbe.
Figure 9: Deflection increase factor due to the effect of vertical earthquake load.



Slika 10: Največji moment (M_{max}) zaradi potresne obtežbe v odvisnosti od potresne intenzitete za neobremenjene modele (q_k=0).
Figure 10: Maximal internal moment (M_{max}) due to the earthquake load dependent on the earthquake intensity and calculated for models with no variable load (q_k=0).

Limit state	FBC 300 cm		TIC 300 cm	
	$\bar{\lambda}_{LS}\ [\cdot\ \mathbf{10^{-3}}]$	$\bar{\lambda}_{LS}^{50}\ [\cdot\ \mathbf{10^{-3}}]$	$\bar{\lambda}_{LS}[\cdot\ \mathbf{10^{-3}}]$	$\bar{\lambda}_{LS}^{50}[\cdot\ \mathbf{10^{-3}}]$
1. End deflection exceedance ($l_k/150$)	0.036	1.82	2.931	137
2. First inelastic deformation	0.103	5.14	0.196	9.74
3. Cantilever collapse due to extreme deflection	0.004	0.20	0.005	0.26
4. Cantilever damage due to uplift	/	/	0.618	30.4

Tabela 2: Frekvence oz. verjetnost prekoračitve mejnega stanja za FBC in TIC model.
Table 2: The limit state frequencies for FBC and TIC model.

collapse for analysed FBC model equals to 4.0•10-6 annually and 2.0•10-4 in 50 year lifetime for the area of Ljubljana. Significant differences occur for TIC model, which has a different critical state mechanism and will most likely be severely damaged due to the cantilever uplift. Probability of TIC model uplift is greater than FBC probability of collapse and equals to 6.2•10-4 annually and 3.0 % in 50 year lifetime. At this point, it must be clear, that TIC model is severely damaged as a result of positive internal moment (cantilever uplift), since load-bearing TI elements are not homogeneous (compressive bearings are not integrated with the rest of the cross-section) and do not contain bottom tension reinforcement. It is assumed that the damage caused by the cantilever uplift reduces the carrying capacity for negative internal moment, which could further deteriorate the seismic response and even lead to collapse. Furthermore, the response spectrum analysis has shown that uplift is more critical for shorter cantilevers. In our case, this would mean that the value of 3% for probability of uplift (300 cm TIC model) can be even higher in the case of shorter cantilevers. The largest difference between the models is certainly a significant increase of deflection for TIC models. The probability of maximum deflection exceedance according to serviceability limit state is displayed only as an orientation value, as earthquake is only a temporary load. Nevertheless, the information is useful in the context of comparison between the two models, since the probability of maximum end deflection exceedance for TIC model is almost 100 times higher.

In addition to the comparison of FBC and TIC models it is necessary to evaluate the analysed seismic risk with the acceptable risk of collapse for structures. It is crucial to understand that there is a certain probability for each structure to collapse. The role of building designers and investors, however, is to determine whether such risk is acceptable or not. The target structure reliability can vary considerably and depends on the intended use of the building, location and many other factors, so it is difficult to choose only one reference value to evaluate the analysed load-bearing TI elements in seismic-prone areas. In spite of these facts we have relied on the acceptable seismic risk provided by the Joint Committee on Structural Safety [JCCS, 2001]. JCCS regulates a seismic target reliability index 3.3 per year for medium important buildings, which corresponds to 2.4% in 50 years. If we compare the results with this value we can conclude that the FBC seismic risk (0.02%) is much smaller than the acceptable level of seismic risk. The latter means that FBC models are appropriate and can be used despite the negative effects of seismic load uncertainty. However, for TIC model the probability of uplift (severe damage) is 3.0% in 50 years, which is more than the target reliability index given in [JCCS, 2001] and from this point of view not acceptable. A preventive measure to include bottom tension reinforcement, which was analysed for FBC models, has proved to be very efficient, so it is advisable to implement bottom reinforcement for precast load-bearing TI elements (TIC models) as well. Negative consequences of cantilever uplift can be effectively reduced by changing the precast elements, providing also the bottom reinforcement.

Conclusion

The results of the study indicate that the existing thermally insulated cantilevers (TIC models) expose limited earthquake safety in comparison with conventional reinforced concrete (RC) fixed base cantilevers' (FBC models). To reduce the seismic risk of TIC models some changes of precast load-bearing TI elements are necessary – tensile reinforcement in the bottom part of the cross-section (half of the upper reinforcement) should be added. The performed analyses proved that the fundamental period of vibration for TIC models is about 2.5 times larger than the FBC models period. Such increase is a consequence of rotational flexibility of the load-bearing TI element at the fixed end of the TIC model. Due to the increased rotational flexibility, TIC models expose large deflections already for vertical static loads, which can exceed the serviceability limit state ($w_{max}=l_k/150$) for cantilever lengths between 300 and 400 cm. In the case of seismic load, the cantilever deflection increases and reaches up to 2 times higher values (for seismic intensity of 0.25 g) or up to 3 times higher values (for 0.45 g) in comparison with static deflection. In addition, the damage of the cantilever can be caused by vertical seismic load, if the bending strength is exceeded, or in special cases, due to the cantilever uplift. For TIC models carrying capacity for negative moment (tension stresses on the top side) can be exceeded for seismic intensity equal to or greater than 0.35 g. On the other hand the critical state of cantilever uplift can be observed already for seismic intensity 0.25 g (tension stresses on the bottom side) and is more likely for shorter cantilevers – the influence of static loads is smaller for short models. In other cases, when the seismic intensity is lower than 0.25 g, the ultimate limit state for vertical static loading is more critical. The latter was confirmed with seismic risk assessment for nonlinear FBC and TIC model in length 300 cm. Probability of severe damage is higher for the analysed TIC model (3.0% in 50 years), which is much more (150 times) than for the FBC model and the acceptable level of seismic risk.

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