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EFFECT OF RESISTANCE CRITERIA ON IMPERFECTION SENSITIVITY ASSESSMENT IN GNIA ANALYSIS

Summary: Eurocode requires the consideration of geometric imperfections. Imperfections can be considered directly, as is the case in GNIA and GMNIA analyses. Recent studies propose various methods for obtaining the governing imperfection, yet they rarely justify the criterion used to determine the buckling resistance from the nonlinear load-displacement curve, nor do they consider how the choice of criterion affects the assessed imperfection sensitivity. This paper presents criteria for determining buckling resistance in GNIA analysis, applied to and compared on two frames whose imperfections were modelled directly using an exhaustive search of imperfection combinations. The results show that the choice of criterion does not merely affect the assessed resistance value, it fundamentally changes the identification of the governing imperfection and its shape, with different criteria potentially identifying entirely different combinations as the most unfavourable. Consequently, the assessed imperfection sensitivity of the frame also depends significantly on the chosen criterion.

Keywords: Eurocode, stability, nonlinear analysis, imperfections

UTICAJ KRITERIJUMA NOSIVOSTI NA PROCENU OSETLJIVOSTI NA IMPERFEKCIJE U GNIA ANALIZI

Rezime: Evrokod propisuje obavezno razmatranje geometrijskih imperfekcija. Imperfekcije se mogu uzeti u obzir direktno što se vezuje za GNIA i GMNIA analize. Novija istraživanja predlažu različite metode za dobijanje merodavne imperfekcije, ali retko obrazlažu kriterijum za očitavanje nosivosti na izvijanje sa nelinearne krive sila-pomeranje, niti razmatraju uticaj izbora kriterijuma na procenjenu osetljivost na imperfekcije. U radu su prikazani kriterijumi za određivanje gubitka stabilnosti za GNIA analizu, primenjeni i upoređeni za 2 rama kod kojih su imperfekcije direktno modelirane iscrpnim pretraživanjem kombinacija imperfekcija. Rezultati pokazuju da izbor kriterijuma ne utiče samo na vrednost procenjene nosivosti, već suštinski menja identifikaciju merodavne imperfekcije i njen oblik, različiti kriterijumi mogu identifikovati potpuno različite kombinacije kao najnepovoljnije. Posledično, i ocena osetljivosti rama značajno zavisi od odabranog kriterijuma.

Ključne reči: Evrokod, stabilnost, nelinearna analiza, imperfekcije

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1. INTRODUCTION

Stability design is a key calculation for steel structures because of the use of slender members that are sensitive to imperfections. For this reason, accurate modelling of imperfections is extremely important, since they significantly affect the distribution of internal forces and the ultimate resistance of the structure. Imperfections in structures can be taken into account either directly or indirectly. Indirect modelling of imperfections is the most widely used and involves accounting for imperfections through the European buckling curves. Another method mentioned in the literature is the reduction of member stiffness, in which imperfections are accounted for by reducing the modulus of elasticity by a reduction factor. Direct modelling of imperfections involves the use of advanced numerical analyses: geometrically nonlinear analysis with imperfections (GNIA) and geometrically and materially nonlinear analysis with imperfections (GMNIA). Direct modelling of imperfections involves the introduction of equivalent geometric imperfections in one of three ways: through measured imperfections of the member, through an equivalent horizontal load, and through a linear buckling analysis using the buckling mode shape or a combination of buckling mode shapes. Most researchers have modelled a large number of imperfections for a single frame in order to assess the influence of individual imperfections on the limit load estimate of the structure [1], [2]. By such a procedure, a large number of nonlinear load-displacement curves is obtained, but the question arises of how to determine the limit load estimate from a nonlinear curve, particularly in situations where the limit load estimates are very close, since it is then difficult to assess whether these differences are truly significant or are a consequence of the choice of criterion for determining loss of stability.

prEN 1993-1-6 [3] provides four stability criteria, C1-C4, used to determine the buckling resistance in nonlinear analyses. However, these criteria are elaborated in detail only in the part of the Eurocode dealing with the stability of shell elements, and no provisions are given for beam elements; nevertheless, these criteria are also used in literature for other types of structures. prEN1993-1-6 [3] defines the buckling resistance as the smallest value obtained by applying three criteria: C1 – the maximum load on the curve; C2 – the load at bifurcation, if it occurs before the maximum; and C3 – the largest tolerable deformation, if it is reached before bifurcation or the maximum load. The C4 criterion is set apart in the Eurocode from the first three and is associated with GNIA analysis and is defined as the load at which the equivalent stress at the most stressed point reaches the yield stress. In addition to these methods from the Eurocode, industrial standards such as ASME prescribe graphical criteria for determining the ultimate resistance. Of the prescribed criteria, two criteria [4] are applicable to GNIA analysis: TES (Twice Elastic Slope) and TI (Tangent Intersection).

The aim of this paper is to assess, on the example of two frames whose imperfections were modelled directly by applying the Imperfection Combination Study (IMC) method [1], that the assessment of imperfection sensitivity is fundamentally dependent on the criterion used to determine the limit load estimate from the nonlinear load-displacement curve. By systematically comparing multiple limit load criteria across all imperfection combinations, the paper shows that different criteria not only yield different limit load estimate, but identify different imperfection shapes as governing and produce substantially different conclusions regarding the degree of imperfection sensitivity. The reliability of the sensitivity assessment is therefore inseparable from the choice of criterion.

2. THEORETICAL BACKGROUND: GNIA ANALYSIS AND GEOMETRIC IMPERFECTIONS

In the stability design of steel structures, GNIA analysis is important because it allows the initial imperfections to be explicitly included in the model, with the analysis then carried out on the deformed structure. GNIA analysis is useful not only for determining the buckling resistance of an imperfect structure, but also for comparing the influence of different imperfections and identifying which of them gives the most unfavourable structural behaviour.

2.1. GNIA ANALYSIS

EN 1993-1 [3] provides seven types of analysis: linear elastic analysis (LA), linear elastic bifurcation analysis (LBA), geometrically nonlinear elastic analysis (GNA), materially nonlinear analysis (MNA), geometrically and materially nonlinear analysis (GMNA), geometrically nonlinear elastic analysis with imperfections (GNIA) and geometrically and materially nonlinear analysis with imperfections (GMNIA). GNIA analysis is a computational analysis that takes imperfections into account using a linear elastic material law. The only source of nonlinearity is geometric: the equilibrium conditions are established on the deformed configuration, and the effect of deformations on the distribution of internal forces is fully accounted for. Imperfections are taken into account such that they have the most unfavourable effect on the resistance under a given loading, which leads to the need to assess the sensitivity to different imperfections in order to determine the governing one. The essence of GNIA analysis is that the calculation is not carried out on the initial, undeformed geometry; instead, the equilibrium conditions are established on the deformed structure. The load is applied incrementally, and after each step the geometry of the structure is updated so that the new deformations influence the next step of the analysis. In this way, GNIA captures second-order effects, that is, the additional moments and the changes in internal forces that arise from the deformed shape of the structure. For this reason, this analysis is particularly suitable for systems in which small initial geometric deviations cause a significant change in behaviour and where it is necessary to assess the sensitivity of the structure to imperfections.

2.2. GEOMETRIC IMPERFECTIONS IN STEEL FRAMES

EN 1993-1-1 [5] requires that both global and local imperfections be taken into account and specifies three ways of directly incorporating imperfections into a structural model: through measured imperfections of the actual member, through an equivalent horizontal load, or through a linear buckling analysis using either the buckling mode shape or a combination of several mode shapes. Imperfections are taken into account such that they exert the most unfavourable effect on the load-carrying capacity under a given loading, leading to the need for a sensitivity assessment to various imperfections in order to determine the governing imperfection shape.

Shayan [2], [6] argues that current methods of directly defining imperfections are overly conservative, given that imperfections are random in amplitude, and proposes a new probabilistic method based on superposing a chosen number of buckling mode shapes. For unbraced planar frames, a linear combination of the first three modes is recommended, as this gives results close enough to those obtained with six modes, whereas for braced planar frames the first six modes are recommended instead. Liu [7] extends the work of Shayan [8], proposing a method for incorporating geometric imperfections into 3D frames composed of both hot-rolled and cold-formed steel sections. By minimising the error between a linear combination of modes and randomly generated geometric imperfections based on experimental data, both the appropriate number of modes and their amplitudes were established, based on a parametric study of eight sway frames and seven non-sway (braced) frames.

Slack et al. [1] developed a method EM3-B for defining initial imperfections in planar frames, based on buckling mode shapes obtained from a linear buckling analysis. The buckling modes first need to be separated into sway and non-sway shapes, each scaled by its own amplitude. The method combines the first sway mode with every non-sway mode whose critical load factor is below 25 ($\alpha < 25$), intentionally leaving out the remaining global sway modes so that the imperfection keeps a single, consistent direction rather than having that direction cancelled out by higher modes. This captures the first global mode, which governs the overall sway of the frame, except where a horizontal load is present, in which case the sway direction is instead dictated by the direction of that load. To verify the EM3-B method, Slack used the IMC method, which provides the most unfavourable imperfection shape. In the present study, the IMC method is used as a basis, with two modifications described in Section 3.

3. NUMERICAL ANALYSIS

Studies investigating the imperfection sensitivity of frames [1], [2] have analysed various frames, on the basis of which certain methods for the direct definition of imperfections have been proposed. In this paper, only two frames are presented (Figure 1). The frames are of steel grade S355. Frame 1 has a span of 10 m and a height of 10 m, with a vertical load at the node of 4610 kN, and horizontal load at the node of 92.2 kN. Frame 2 has a span of 9.14 m and a height of 4.57 m, with a vertical load at the node of 1550 kN, while the horizontal load has an intensity of 155 kN.

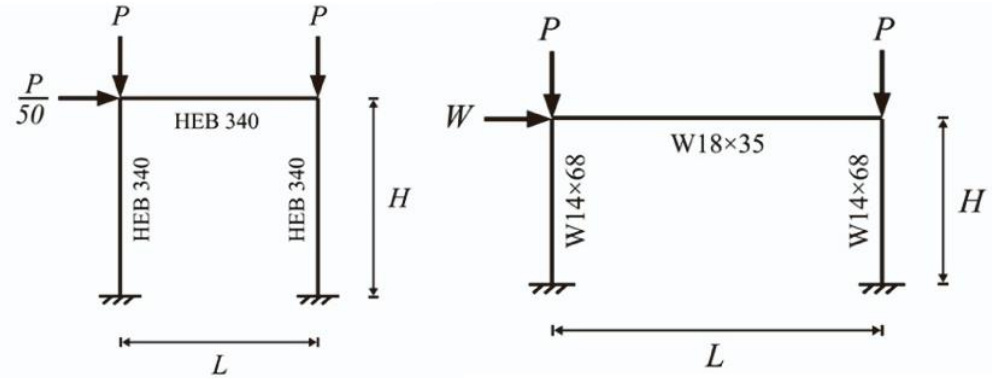

Figure 1: Frame 1 (left) and Frame 2 (right)

Both frames were modelled using beam finite elements with an adopted element length of 0.5 m for Frame 1 and 0.508m for Frame 2, resulting in 20 finite elements per column and per beam for Frame 1, and 9 finite elements per column and 18 per beam for Frame 2. Both frames are planar, with fixed base supports and rigid beam-to-column connections. The applied loading was scaled by the first critical load factor obtained from LBA. In the GNIA analysis, the horizontal displacement of the node at which the horizontal force is applied was tracked for both frames.

Table 1: Frame 1 and Frame 2 properties

	Frame 1	Frame 2
Span [m]	10	9.14
Height [m]	10	4.57
Boundary conditions	Fixed	Fixed
Vertical load [kN]	4610	1550
Horizontal load [kN]	92.2	155
Beam cross section	HEB 340	W18X35
Column cross section	HEB340	W14X68
Steel	S355	S355
Beam bow imperfection [mm]	22.67	12.80
Column bow imperfection [mm]	22.67	6.4
Sway imperfection [mm]	25	11.43
Number of imperfections	128	128

The imperfections were determined using the Imperfection Combination Study (IMC) method, proposed by Slack et al. [1] as a method for determining the most unfavourable imperfection. In this method, different variations of the directions and shapes of global and local imperfections are used, in various mutual combinations, with the aim of obtaining a set of variously deformed structural models. The global sway imperfection amplitude at each storey is defined as H/400, while the member bow imperfection amplitude is defined as the greater of αL/150 and L/1000, where α is the imperfection factor corresponding to the appropriate buckling curve of the cross-section. In the original method, each imperfection is characterised by a direction vector whose elements take values of either −1 or +1, corresponding to the two possible directions of each sway and bow imperfection component. In this paper, the IMC method was modified in order to adapt the model to the realistic behaviour of the structure. The first modification is the addition of a local imperfection (bow imperfection) in the shape of a full wave, in addition to the existing half wave, so as to cover a wider range of imperfections. The bow imperfection of each member is discretised at all intermediate nodes along the member length using either a half-sine wave shape, sin(πx/L), representing a single-curvature bow, or a full-sine wave shape, sin(2πx/L), representing an S-shaped double-curvature bow, where x is the position along the member and L is the member length. The second modification is the omission of the global imperfection of nodes in opposite directions, since such an imperfection implies an elongation of the beam that is not realistic. For a single-bay single-storey portal frame with two columns and one beam, the total number of imperfection combinations is determined by the number of independent parameters and their possible states. The sway direction at the joints can take two values (±1), resulting in 2 sway combinations. Each column and beam bow imperfection can take one of four forms: a half-wave or a full-wave imperfection, each applied in either of two directions resulting in $4^3 = 64$ bow combinations. The total number of imperfection combinations is therefore 64 × 2 = 128, all of which were evaluated for each of the two frames considered in this study. Accordingly, an equal number of nonlinear curves was obtained, which serve as the basis for assessing the influence of the different criteria for determining the sensitivity to imperfections on this frames.

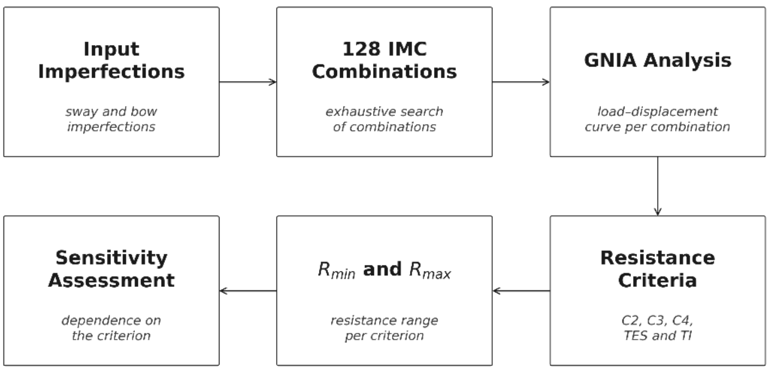

Figure 2: Methodological flowchart for imperfection sensitivity assessment

The nonlinear analysis was carried out in the academic software Matrix3D [8]. The tangent stiffness method was used, which computes the solution incrementally. Based on the current state at the integration points, this method determines the increment of displacements and strains and updates the stiffness matrix of the system accordingly. If the internal forces in the elements do not balance the external load, the software computes the residual (unbalanced) forces and initiates an iterative Newton-Raphson procedure until convergence. In addition, when pronounced nonlinearities arise in the system, the software activates the arc-length method, which introduces an additional constraint equation into the global system, enabling stable tracking of the structural behaviour. Although Matrix3D is capable of material nonlinear analysis, only geometrical nonlinearity with a linear elastic material law was activated in the present study.

4. CRITERIA FOR ASSESSING IMPERFECTION SENSITIVITY

prEN 1993-1-6 [3] provides four stability criteria, C1-C4, used to determine the buckling resistance in nonlinear analyses, but not all of them are applicable to GNIA analysis. Other methods not mentioned in the Eurocode also appear in the literature. Mackenzie and Li [4] studied the problem of computing the plastic collapse load in pressure vessels using nonlinear analysis (GMNIA), giving two criteria (Figure 4) that could also be applied when determining the buckling resistance in GNIA analysis: the Twice Elastic Slope (TES) criterion and the Tangent Intersection (TI) criterion.

4.1. prEN 1993-1-6

prEN 1993-1-6 [3] is the part of the Eurocode covering shell structures, and it provides the most detailed procedure for determining the buckling resistance from GNIA and GMNIA analyses (Figure 3), and it is therefore used as a reference for other types of structures as well. The buckling resistance is denoted RGMNIA and is taken as the smallest value of the three criteria (C1-C3).

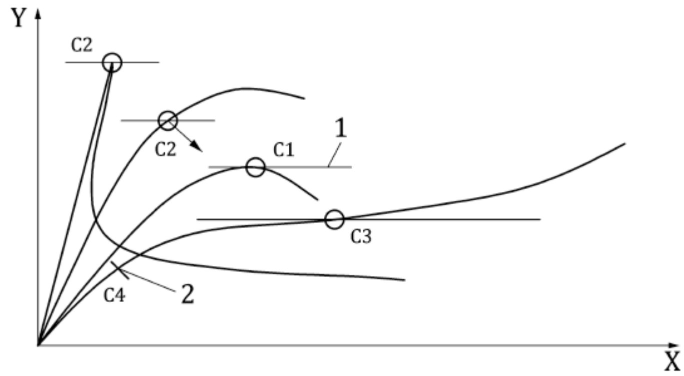

Figure 3: Definition of buckling resistance from a GMNIA analysis

C1 is the criterion that defines the buckling resistance as the maximum load factor on the load–deformation curve, but it can be applied only when the curve has a clearly pronounced maximum, which is rare in GNIA analyses and more common in GMNIA analyses, and for this reason this criterion was not used in this paper. The C2 criterion takes the load factor at bifurcation as governing, but only in cases where the bifurcation load factor is smaller than the maximum load factor. For this criterion, the check is performed through the eigenvalues of the tangent stiffness matrix. The third criterion, C3, defines the buckling resistance by means of the largest tolerable deformation and is used as governing only in cases where the load at which the largest tolerable deformation is reached is smaller than the load for criteria C1 and C2. The Eurocode particularly emphasises that criterion C3 is intended more for the serviceability limit state than for the conventional ultimate limit state, but that it can be useful for structures whose behaviour is elastic and which are nevertheless subject to very large deformations without a clearly defined collapse. If the limiting deformation value is not known, the Eurocode proposes adopting as the limit the point at which the largest local rotation of the shell surface reaches β_{lim} =0.1 rad. For structures that exhibit pronounced yielding, the plastic deformation limit can alternatively be used. To apply criterion C3 to beam elements, it is necessary to select a governing deformation and its value (horizontal displacement of the node, interstorey displacement (drift), or relative rotation of the member ends). Since Eurocode does not prescribe a specific deformation limit for beam element models of steel frames, a practical limit was adopted from seismic design standards. Codes such as ASCE-41 [9] and prEN 1998 [10], [11] specify an interstorey drift ratio limit of 4% as the limiting value for the prevention of global collapse due to nonlinear geometric effects. This limit was adopted in the present study as a practical reference value for determining the buckling resistance from the GNIA analysis of the considered frames. The C4 criterion is set apart in the Eurocode because, unlike the first three, it is associated not with GMNIA analysis but with GNIA analysis, that is, the case in which there is no material nonlinearity, which is the subject of the research in this paper. For that case, a conservative estimate of the buckling resistance is used, RGNIA, which is obtained as the load factor at which the equivalent stress at the most stressed point of the structure's surface reaches the design value of the yield stress, $f_{yd} = f_{yk}/\gamma_{M0}$. The Eurocode emphasises that this criterion is conservative because it accounts only for the onset of yielding at the surface of the elements, thereby neglecting the reserve of plastic resistance.

4.2. TWICE ELASTIC SLOPE (TES CRITERION) AND TANGENT INTERSECTION (TI CRITERION)

Stability loss can be identified not only through the singularity of the stiffness matrix but also through the change in tangent stiffness. The stiffness-drop criterion represents a practical numerical approach for determining the limit load estimate from the nonlinear force–displacement curve, particularly in cases where the curve has no clearly pronounced force maximum, as is the case in GNIA analysis. The Twice Elastic Slope (TES) criterion [4] is applied by constructing, from the coordinate origin, a line whose slope is twice that of the elastic part of the diagram. The point at which this line intersects the nonlinear curve is defined as the failure load. For the Tangent Intersection (TI) criterion, two lines are constructed. The first line starts from the coordinate origin and corresponds to the slope of the elastic part of the diagram, while the second is the tangent to the plastic part of the diagram. The intersection point of these two lines is projected onto the vertical axis and defines the failure load. A disadvantage of the TI criterion is that it is unclear where the tangent to the plastic part of the curve should be drawn, since its slope is not constant. Within the academic software Matrix3D, the initial elastic tangent was determined by least-squares regression through the origin using the first five points of the load-displacement curve, thereby capturing the linear elastic stiffness of the frame, while the second tangent was fitted by linear regression to the final fifteen points of the curve, forming a tangent to the plastic part of the curve.

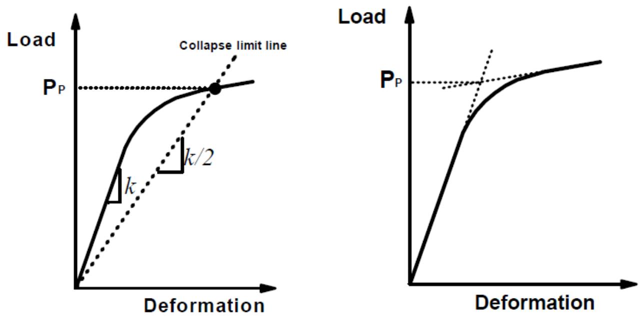

Figure 4: TES criterion and TI criterion

5. RESULTS AND DISCUSSION

The C1 criterion specified in the Eurocode was not applied here, as the nonlinear force–displacement curves obtained from the GNIA analyses show no clearly defined maximum. The C2 criterion was verified directly within the nonlinear solver implemented in the academic software Matrix3D, by computing at each load step the smallest eigenvalue of the tangent stiffness matrix and tracking its variation throughout the steps. Buckling was identified as the step at which the eigenvalue first changes from positive to negative or vice versa, which corresponds to the singularity of the tangent stiffness matrix. For Frame 1, no bifurcation was detected because the smallest eigenvalue tends to zero asymptotically without a change of sign; for this reason the C2 criterion was not considered for Frame 1, whereas for Frame 2 a bifurcation was detected and this criterion was therefore considered. The C3 criterion was analysed in this paper, adopting a limit of 4% of the maximum interstorey drift as the limit defining failure. The C4 criterion was verified by computing, at each load step, the internal forces at the ends of every member, from which the stress at the outermost point of the cross-section was calculated. The maximum stress was computed for the whole structure at each step, and the load factor at which the yield strength of the steel was first reached in any cross-section was obtained. prEN 1993-1-6 specifies that the resistance RGMNIA is determined as the smallest load factor obtained from the three criteria C1, C2 and C3. The C4 criterion is treated separately, as an alternative method; however, clause 9.8.2 (16) of prEN 1993-1-6 states that the C4 criterion is used when RGMNIA is to be estimated through the simpler GNIA analysis, unless a bifurcation has been detected earlier. For this reason, the C2, C3 and C4 criteria are compared here together with the TES and TI criteria. Tables 2 and 3 present the results for all five criteria for the least critical and the governing imperfection, where the difference is calculated as (R_{max} - R_{min}) / R_{min} × 100%. While Figure 5 shows the results of the nonlinear analyses for all imperfection shapes and for four criteria (C3, C4, TES and TI), together with the critical points.

Table 2: Results for Frame 1

Criterion	Highest limit load (is associated with least critical imperfection) [kN]	Lowest limit load (is associated with governing imperfection) [kN]	Difference [%]	Least critical imperfection (index)	Governing imperfection (index)
C3	11704.9	10857.1	7.81	64	107
C4	8187.9	6047.4	35.4	32	71
TI	9940.7	9755.5	1.9	95	28
TES	11352.2	11217.6	1.2	125	42

Table 3: Results for Frame 2

Criterion	Highest limit load (is associated with least critical imperfection) [kN]	Lowest limit load (is associated with governing imperfection) [kN]	Difference [%]	Least critical imperfection (index)	Governing imperfection (index)
C2	25627.2	25438.3	0.75	100	39
C3	27553.5	25649.1	7.43	64	107
C4	4206.3	3801.9	10.64	17	71
TI	26106.6	25950.5	0.6	120	33
TES	23116.1	22997.7	0.52	54	97

The imperfection sensitivity differs significantly between the various methods. In these examples the C2, TI and TES criteria give a difference between the least critical and the governing imperfection of less than 2%, whereas the C4 criterion gives the largest difference — as much as 35.4% for Frame 1 and 10.6% for Frame 2 (Tables 2 and 3). This clearly shows that the choice of criterion for determining the limit load estimate strongly affects the assessment of the structure's sensitivity to imperfections. Figure 6 shows the imperfection patterns referred to in Tables 2 and 3 for criteria C3 and C4.

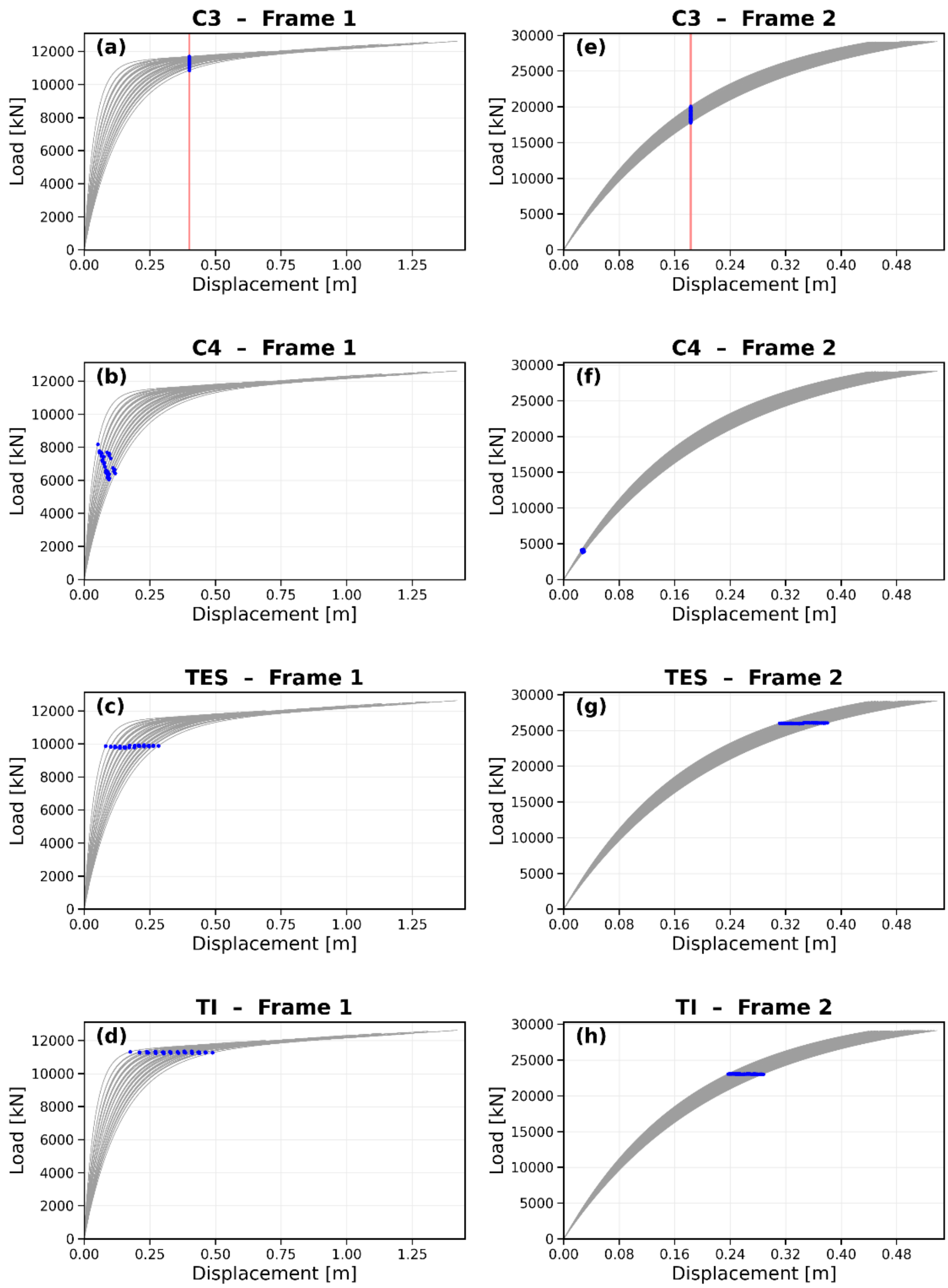

Figure 5: Load-displacement curves obtained from GNIA analyses for all imperfection patterns and selected resistance criteria

The geometric arrangement of the limit load points (blue markers) on each load–displacement family reveals why, for the two frames analysed here, the four criteria differ so strongly in their sensitivity to the imperfection shape (Figure 5). For C3 criteria the limit load is read at a fixed horizontal displacement, so its points align along a vertical line. At that displacement the response curves of the individual imperfection combinations are still spread apart in load, and the vertical cut therefore crosses a wide range of load levels; this is why C3 criteria returns markedly different limit loads from one imperfection to another. For the C4 criterion for each imperfection, first yield occurs at a different displacement and a different load, so the points do not line up. These points lie on the steep, rising part of the curves, where a small change in displacement gives a large change in load. As a result, the failure loads vary widely between imperfections.

The tangent-based criteria behave in the opposite way. For TES and TI the points line up almost horizontally, on the part of the curves where the slope has already become small so all points sit at almost the same load even though their displacements differ. As a result, the failure loads of all 128 imperfection combinations fall into a very narrow band, and the differences between them are negligible. The direction in which the points line up: vertical for C3, inclined for C4 and horizontal for TES and TI therefore reflects how much the imperfection shape affects the predicted limit load estimate in the analysed frames.

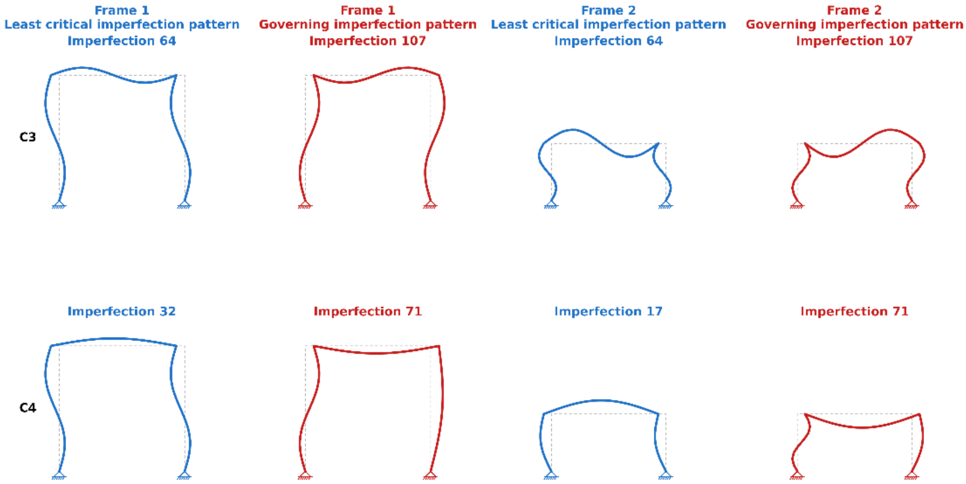

Figure 6: Least critical imperfection pattern and governing imperfection pattern for Frame 1 and Frame 2 (C3 and C4 criteria)

The governing imperfection according to criteria C3 and C4 for both frames (107 and 71) is characterised by a horizontal imperfection of all nodes in the same direction. Such an imperfection shape directly increases the geometric nonlinearity effects (P-Δ effect), because the horizontal imperfection of the column acts in the same direction as the horizontal displacement of the structure caused by the horizontal load, thereby increasing the bending moments in the column and accelerating the stability loss. In contrast, the least critical imperfections have the direction of the global imperfections opposite to that of the horizontal force, which reduces the P-Δ effects. For the C2, TES and TI criteria, the difference in limit load estimate between the least critical and governing imperfection shapes is less than 2%, indicating that these criteria were not sufficiently discriminative for ranking imperfection shapes for the two frames analysed here. The C4 criterion depends directly on the stresses in the critical cross-sections, which are highly sensitive to whether the imperfection increases or decreases the bending moment in the column; this explains the substantially larger differences in limit load estimate between the least critical and the governing imperfection shapes (Tables 2 and 3). For the C4 criterion, the Eurocode emphasises that it gives the most conservative results, which is also evident in the case of these frames, since the lowest limit load estimates were obtained precisely by this method. An analysis of the first 20 worst imperfection shapes showed that the difference between the most critical case and the 20th is less than 5%, which means that a single governing imperfection shape cannot be identified; rather, there is a whole group of mutually similar imperfections that give an identical limit load estimate.

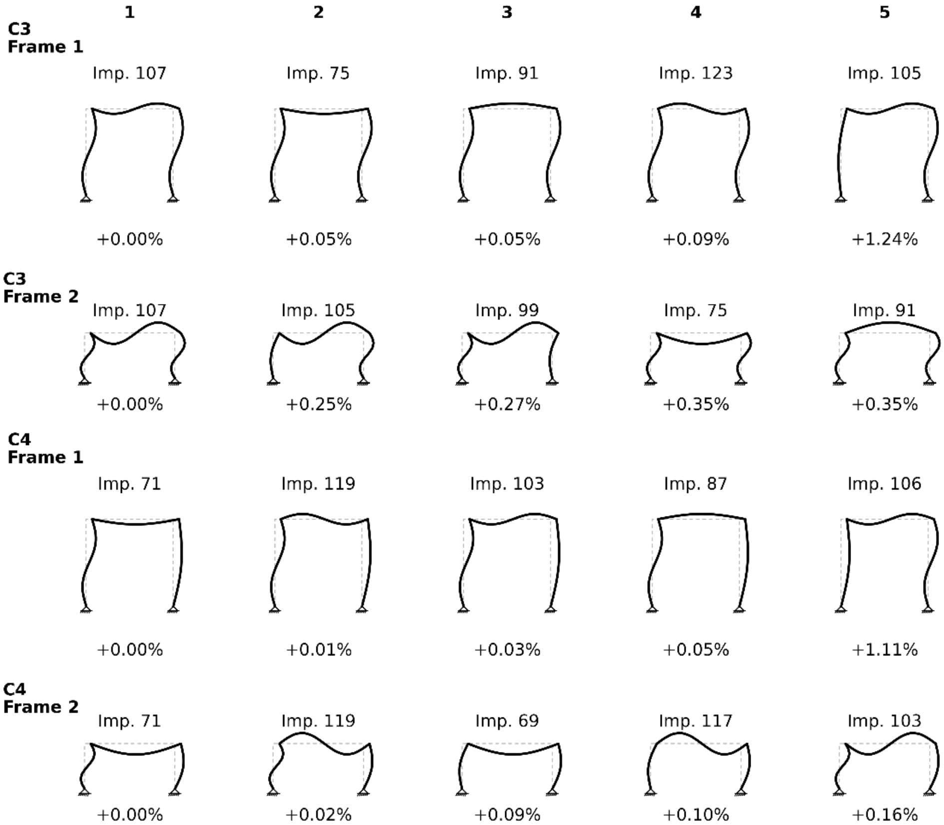

Figure 7: First 5 governing imperfection patterns for Frame 1 and Frame 2 (C3 and C4 criteria)

Figure 7 shows the imperfection shapes for the first 5 worst cases for the C3 and C4 criteria for both frames, together with the percentage differences in loads relative to the worst case. For the C3 criterion, the first 20 most critical imperfection shapes have in common that the global imperfection acts in the direction of the applied force, and in 14 of the 20 cases the columns have a local imperfection in the form of a full-wave, while the beam has no consistent influence. An analysis of the first 20 worst imperfection shapes for the C4 criterion leads to the same two conclusions as for the C3 criterion: that the governing direction of the global imperfection is in the direction of the force, and that the beam imperfection has no significant influence on the limit load estimate. The difference is that for the C4 criterion an asymmetric arrangement of the local column imperfections is dominant (one column has a bow imperfection in the form of a full-wave, while the other has a bow imperfection in the form of a half-wave) among the first 20 worst imperfections. For the C4 criterion, 18 of the 20 worst imperfections are the same combinations (identical index and shape) in both frames. From this it is clear that the conclusions drawn are not entirely the same, depending on the criterion used for determining the limit load estimate.

6. CONCLUSION

- The results show that the assessment of imperfection sensitivity in GNIA analysis depends strongly on the criterion used to extract the representative resistance from the nonlinear load-displacement curve

- For the analysed frames, the C3 and especially the C4 criterion produced the largest differences between imperfection cases, whereas the C2, TES and TI criteria led to differences below 2%. This indicates that the latter criteria are not sufficiently discriminative for ranking imperfection patterns in the considered examples.

- The C4 criterion gave the lowest resistance values and the largest variation because it is governed by the first attainment of yield stress in the most stressed cross-section. Therefore, it should be interpreted as a conservative elastic-limit indicator rather than as a direct measure of global buckling resistance

- The results also indicate that a unique governing imperfection shape cannot be identified. Instead, a group of similar imperfection patterns produces nearly identical resistance values. The most unfavourable cases are generally characterised by global imperfections acting in the direction of the applied horizontal displacement, thereby amplifying second-order effects

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