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Displacement- and Performance-Based Seismic Design for Sustainable Earthquake Resistant Concrete Construction

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ABSTRACT

A seismic design procedure is proposed for specific and measurable performance of concrete structures for at least two seismic hazard levels. It tailors capacities to seismic demands over the structure, allowing significant savings in materials compared to conventional opaque design codes with prescriptive and wasteful detailing. The procedure is described in detail as a sequence of steps. Design expressions are given for carrying out these steps in bridges with integral deck and piers or buildings. Detailing of members for ductility is based on a transparent explicit verification of inelastic deformation demands against capacity limits. An application to eight bridges shows major savings in steel without loss in seismic performance.

INTRODUCTION

Design of structures for sustainability should entail, among other, concerted efforts to use as little material as absolutely necessary for the target performance of the structure. Such efforts should be undertaken, of course, by the individual designer of each structure. Besides, codes and standards should promote design for minimum material quantities and explicitly include a requirement to estimate and reduce the structure's carbon footprint. Unfortunately current design codes are far from there. This is the case in particular with seismic design according to current codes, which is still force-based, with reduction of the lateral forces derived from the elastic spectrum by the behavior (or force reduction) factor. Prescriptive member detailing rules are employed, to provide the ductility associated with the value of the behavior factor used in the design. This approach is opaque as far as seismic performance is concerned, but also very wasteful, because the prescriptive detailing rules result in an excess of material for the achieved performance. Performance-based seismic design allows targeting specific and measurable performance under - normally more than one – seismic hazard levels. Moreover, if based on displacements and their derivatives, i.e., deformations, it allows tailoring the capacities to the seismic demands throughout the structure and savings in materials. In this paper a practical procedure is proposed for displacement- and performance-based seismic design of concrete structures (buildings or bridges with integral deck and piers). The concept has evolved from the work of Panagiotakos and Fardis [1999, 2001] for buildings and Bardakis and Fardis [2008] for bridges and draws its tools from [Biskinis and Fardis, 2008].

PROPOSED SEISMIC DESIGN PROCEDURE

Performance levels, requirements and criteria

Structures are designed here to meet the Immediate Use Serviceability Limit State and at least one of two Ultimate Limit States (ULS): (a) Life Safety and (b) Near Collapse. Each Limit State (performance level) is met at its own seismic hazard level, with the seismic action defined for each level via its own 5%-damped elastic spectrum.

The proposed compliance criteria for the three Limit States (performance levels) are:

a) For Immediate Use, nominal yielding at potential plastic hinges may be exceeded by a factor with a value of about 2.0, reflecting a presumed overstrength factor of at least 1.5 in materials and members and certain tolerance of flexural yielding at some sections.
b) At the Near Collapse Limit State, members should stay below an upper-fractile (e.g., a 95%-fractile) of their ultimate flexural deformation, to account for model uncertainty.
c) At the Life Safety Limit States, a safety margin against the above upper-fractile ultimate flexural deformation at member ends should be provided. A safety factor of 1.5 seems appropriate for ordinary buildings, or of 2.0 for bridges.
d) Brittle (i.e., shear) failures of members and their connections should be prevented at both ULSs. A safety factor of 1.25 on shear resistances based on design values of material strengths (nominal strengths divided by the material partial factor) seems appropriate.

A constant safety factor between the limits for Life Safety and Near Collapse in (b) and (c) means that it is normally redundant to check flexural deformations at both ULSs. The Near Collapse ULS governs if the spectral values of its own seismic action at the elastic structure's important natural periods (from Step 5 below for the final effective stiffness) clearly exceed those of the Life Safety action by more than the inverse of the safety factor. Conversely, if they are clearly less. Besides, the shear verifications are carried out only once. In the end the three-tier design may reduce to single tier design in shear and a two-tier one in flexure.

Steps of the proposed design procedure:

Step 1 - Conceptual design and sizing of members: Select a clear and simple structural system, with the maximum feasible regularity of geometry and mass in plan and in elevation. Provide sufficient lateral stiffness near the perimeter in plan to ensure that the period of the lowest primarily transnational mode in two orthogonal horizontal directions is longer than that of a torsional mode. Size the members as follows, for fruition of Step 7 below:

- In buildings, vertical members of the same family (i.e., walls or columns) should have as uniform a cross-sectional depth as possible in each one of the two horizontal directions. If the rotational restraint of members by others in the considered horizontal direction varies among members of the same family, those restrained more by others may be chosen with smaller depth than the rest.
- In buildings, the beam depth should be uniform all-along any plane frame, but may be smaller in frames with shorter spans than in others with longer ones. It should gradually decrease from the base to the roof.
- In bridges, the section and free length of piers integral with the deck should be chosen so that they all present about the same lateral stiffness in each one of the two orthogonal horizontal directions of the seismic action (longitudinal and transverse).
- Member cross-sections should be large enough for their connections to accommodate bond requirements for the intended sizes of bars passing through them or anchored there.

Step 2 - Design for non-seismic actions: Dimension the reinforcement of all members on the basis of the Ultimate and the Serviceability Limit States for all pertinent non-seismic actions, taking into account minimum reinforcement requirements for structures without earthquake resistance. Redistribute ULS moments in horizontal members from supports to mid-spans or vice-versa, as optimal in design for non-seismic actions. In bridges, design also for the ULS at all relevant intermediate stages of construction, taking into account the redistribution of action effects due to creep and losses of prestress, etc., as appropriate.

Step 3 (only in buildings) – Capacity Design against story-sway mechanisms. Unless the walls in a horizontal direction of the building are considered sufficient to preclude a story-sway mechanism, determine the vertical reinforcement of columns so that the sum of their moment resistances around any beam-column joint exceeds (and indeed with a margin of at least 30%) that of the beams framing into the joint in that direction. Base this check on the beam moment resistance from earlier steps and on the axial force due gravity loads concurrent with the seismic action.

Step 4 - Member effective stiffness: Estimate the member effective stiffness, $(EI)_{eff}$, as representative of the elastic stiffness during the response to the corresponding seismic action, using in the calculation the axial force due to the gravity loads concurrent with the seismic action and mean values of material properties from nominal strengths:

- For members that may yield at one or both ends where the member frames into another or into the foundation (i.e., in beams, columns and walls of buildings and in bridge piers), use the secant stiffness to the yield point of the full member between its two ends:

$$EI_{eff} = \frac{M_y L_s}{3\theta_y} \quad (1)$$

where M_y and L_s are the yield moment and the moment-to-shear-ratio ("shear span") at the yielding end of the member and θ_y the chord rotation there (i.e., the deflection of the end of the shear span divided by L_s) at yielding [Biskinis and Fardis, 2008]:

– Rectangular beam/columns: $$\theta_y = \varphi_y \frac{L_s + a_V z}{3} + 0.0014\left(1 + 1.5\frac{h}{L_s}\right) + \theta_{y,slip} \quad (2a)$$

– Walls or hollow rectangular members: $$\theta_y = \varphi_y \frac{L_s + a_V z}{3} + 0.0013 + \theta_{y,slip} \quad (2b)$$

– Circular piers: $$\theta_y = \varphi_y \frac{L_s + a_V z}{3} + 0.0022 \max\left(0; 1 - \frac{L_s}{6D}\right) + \theta_{y,slip} \quad (2c)$$

In Eqs. (2) φ_y is the yield curvature of the end section (from plane section analysis with elastic σ-ε relations till yielding of the tension or the compression chord), a_v a zero-one variable: a_v=0 if the shear force at diagonal cracking, V_{Rc} (i.e., the shear resistance of members without shear reinforcement in Eurocode 2), exceeds the yield force at flexural yielding: $V_{Rc}>M_y/L_s$ and a_v=1, otherwise, z is the internal lever arm and :

$$\theta_{y,slip} = \frac{\varphi_y d_{bL} f_{yL}}{8\sqrt{f_c}} \text{ (with } f_{yL} \text{ and } f_c \text{ in MPa)} \quad (3)$$

the fixed-end rotation of the end section due to pull-out of tension bars (with mean diameter d_{bL}) from their anchorage beyond the member end (in the foundation or the joint). The shear span, L_s, may be taken as 50% of the clear length of columns between beams or beams between columns in the plane of bending, or of bridge piers fixed against rotation by the deck in the plane of bending. In the strong direction of walls in buildings, L_s in a story may be taken as 50% of the height from the wall's base in the story to the top of the wall. If the member cantilevers within the plane of bending, L_s is its clear length.
The $(EI)_{eff}$-value to be used is the mean from Eq.(1) in the positive and negative directions of bending at the end likely to yield (the mean is also over the two ends if both may yield)
Eqs. (1)-(3) require knowing the longitudinal reinforcement. If this reinforcement is likely to be controlled by the subsequent phases of seismic design instead of Steps 2 and 3

above, and if it is the first time Step 4 is carried out (cf. Step 8 for the iterations), the outcome of Eqs. (1)-(3) should be checked against that of empirical expressions independent of the reinforcement, e.g. [Biskinis and Fardis, 2008]:

$$\frac{(EI)_{\text{eff}}}{E_c I_c} = a\left(0.8 + \ln\left[\max\left(\frac{L_s}{h}; 0.6\right)\right]\right)\left(1 + 0.048 \min\left(50\,MPa; \frac{N}{A_c}\right)\right) \quad (4)$$

where I_c, A_c and h are the moment of inertia, the area and the depth of the section, E_c the Elastic Modulus of concrete, the axial load N is positive for compression, N/A_c is in MPa, and a=0.081 for rectangular columns or circular piers, a=0.1 for beams, a=0.112 for rectangular walls and a=0.09 for members with T-, H-, U- or hollow rectangular section. The larger of the two $(EI)_{\text{eff}}$-values from Eqs. (4) or (1)-(3) should be used. If Step 4 is carried out after Step 7 within iterations towards convergence of the stiffness values, $(EI)_{\text{eff}}$ is calculated only from Eqs. (1)-(3), using as L_s the moment-to-shear ratio from the seismic analysis in Step 5 at the end of the member where the moment is largest.

- For bending of bridge decks about the horizontal axis (about which the section and prestressing are asymmetric), $(EI)_{\text{eff}}$ is taken from a moment-curvature diagram as the secant stiffness between: (a) the point of cracking in bending that puts the mean tendon in tension, and (b) the point of first yielding of the non-prestressed reinforcement in the opposite direction of bending (average of secant value at the two nodes of a deck element). For bending of the deck about the vertical axis, $(EI)_{\text{eff}}$ may be taken as 85% of the elastic stiffness of the uncracked gross concrete section.

Step 5 - Linear analysis for the Immediate Use seismic action: Carry out a modal response spectrum analysis for the seismic action after which Immediate Use is desired, using its 5%-damped elastic spectrum and the estimates of effective stiffness from Step 4. The expected maximum value of each seismic action effect is obtained by combining modal contributions via the complete quadratic combination rule and by taking the square-root-of-sum-of-squares of the seismic action effects for the (two or three) individual seismic action components.

Step 6 - Demand-capacity ratios at the Immediate Use seismic action: Compute the ratio:

- of the elastic moment demand, *D*, taken equal to the seismic moment from Step 5 plus the one due to the concurrent gravity loads, to
- the corresponding design resistance, *C*, (i.e., with design values of material strengths).

at any section where a member is connected to another having stiffness in a plane of bending normal to the vector of the moment in question.

Step 7 - Tailoring of flexural capacities to demands for uniformly distributed inelasticity: Increase the longitudinal reinforcement at all locations where plastic hinges are intended to develop, so that their *D*/*C* ratios are as uniform as possible:

- within the following families of such locations:
  - the wall base sections in buildings with a wall or dual (frame-wall) structural systems,
  - the base sections of bridge piers or columns in buildings with a frame or dual system,
  - the end sections of building beams connected to stronger columns (i.e., whose sum of moment resistances above and below a joint exceeds the corresponding sum in the beams framing into the joint),
  - the ends of building columns connected to stronger beams (whose sum of moment resistances across a joint exceeds that of the columns above and below the joint),
  - the end sections of bridge piers which are fixed against rotation by the deck in the plane of bending, as well as
- between different ones among the above families, as relevant.

The apparent aim of this step is to promote simultaneous yielding under the Immediate Use seismic action at as many locations as possible, but its deeper motivation is to reduce overstrengths and uniformize the distribution of inelasticity among members of the same family or among different families in stronger earthquakes. Compliance criterion no (a) above for the Immediate Use Limit State suggests a target *D*/*C*-value in each family around 2. However, important plastic hinge locations may come out of Step 6 with *D*/*C* values well below this target. If the *C*-value at such a location is governed by Step 2, it cannot be reduced for increasing *D*/*C* towards the target value of 2.0. Unless such low *D*/*C* values are sporadic and do not cast doubt about the prevailing plastic mechanism, we may achieve the target *D*/*C*-value at these locations by raising the seismic action for Immediate Use (as well as the capacities at all other locations, so that $D/C \approx 2$ there under the increased seismic action level).

Step 8 - Iterations with updated stiffness values. Repeat Steps 3-7, using everywhere the longitudinal reinforcement from Step 7. The change in stiffness may affect the demands and partly undo the harmonization achieved in Step 7. So iterations may be needed through Steps 3-8 until satisfactory convergence. Depending on the progress towards convergence, we may have to overshoot in Step 7 (i.e., increase the low *C*-values more than required for the target *D*/*C*-value), in order to harmonize in the end the *D*/*C* values over all potential plastic hinges.

Step 9 - Analysis for the Life Safety or/and the Near Collapse seismic actions. Determine the chord rotations at the ends of all members due to all relevant simultaneous components of the Life Safety or the Near Collapse seismic action, whichever seems most critical according to the reasoning in the last paragraph before Step 1 above. Thanks to the uniform distribution of inelasticity promoted through Step 7, modal response spectrum analysis may well be used, with the 5%-damped elastic spectrum of the seismic action in question. If this spectrum is proportional to that of the Immediate Use seismic action over the range of natural periods considered, seismic action effects from Step 5 are scaled-up by that proportionality constant and added to those due to the concurrent gravity loads. Alternatively, nonlinear dynamic analysis may be carried out both for the Life Safety and the Near Collapse seismic actions.

Step 10 - Member detailing for the chord rotation demands: Check the chord rotation demands, θ_{Ed}, from Step 9 against the chord rotation capacities at member ends where plastic hinges are intended or likely to form:

$$\theta_{Ed} \leq \theta_{Rd} = \theta_{uk,0.05}/\gamma_R \tag{5}$$

where θ_{Rd} is the design value of the member chord rotation capacity, γ_R a safety factor against exceedance of the ultimate chord rotation with values γ_R.= 1.0 if θ_{Ed} comes from the Near Collapse seismic action and γ_R.= 1.5 or γ_R.= 2.0 for ordinary buildings or bridges, respectively, if it refers to the Life Safety level; $\theta_{uk,0.05}$ is the 5%-fractile value of the ultimate chord rotation, obtained by dividing its mean (expected) value by a model uncertainty factor, γ_{Rd}:

$$\theta_{uk,0.05} = \theta_{um}/\gamma_{Rd}. \tag{6}$$

The expected (mean) value of the ultimate chord rotation at a member end may be found as:

$$\theta_{u,m} = \theta_y + (\varphi_u - \varphi_y) L_{pl} \left(1 - \frac{L_{pl}}{2L_s}\right) + \Delta\theta_{slip,u-y} \tag{7}$$

with θ_y from Eqs.(2), L_{pl} the plastic hinge length, $\Delta\theta_{slip,u-y}$ the post-yield part of the fixed-end-rotation due to slip of longitudinal bars from their anchorage zone past the member end and φ_u, φ_y the ultimate and yield curvature, respectively, of the end section from plane section analysis. For φ_u the simplified parabola-rectangle σ-ε diagram may be used for concrete in compression and a bilinear one with linear strain-hardening for the reinforcing bars. Calculation of φ_u should take into account all possible failure modes (with mode (b) governing over (c) or (d) if the moment resistance of the confined core is more than 80% of that of the full unspalled, unconfined section) [Biskinis and Fardis, 2008]:

(a) rupture of tension bars the full, unspalled section, taken to take place under cyclic loading at strain:

$$\varepsilon_{su,cyc} = (3/8)\varepsilon_{u,k} \tag{8}$$

(b) exceedance of the ultimate concrete strain ε_{cu2}=0.0035 at the extreme compression fibers of the unspalled section;

(c) rupture of tension bars according to Eq.(8) in the confined core after spalling of the cover;

(d) exceedance of the ultimate strain $\varepsilon_{cu2,c}$ of the confined core after spalling, where .

$$\varepsilon_{cu2,c} = 0.0035 + 0.4\frac{\alpha\rho_w f_{yw}}{f_{cc}} \tag{9}$$

with ρ_w: ratio of transverse reinforcement in the direction of bending (or the minimum in the two transverse directions for biaxial bending), f_{yw}: its yield stress and α: the confinement effectiveness factor:

- for rectangular sections: $$\alpha = \left(1-\frac{s_h}{2b_o}\right)\left(1-\frac{s_h}{2h_o}\right)\left(1-\frac{\sum b_i^{\,2}/6}{b_o h_o}\right) \tag{10a}$$
- for circular sections with circular hoops: $$\alpha = \left(1-\frac{s_h}{2D_o}\right)^2 \tag{10b}$$
- for circular sections with spiral reinforcement: $$\alpha = \left(1-\frac{s_h}{2D_o}\right) \tag{10c}$$

with s_h denoting the centerline spacing of stirrups, D_o, b_o and h_o the confined core dimensions to the centerline of the hoop and b_i the centerline spacing along the section perimeter of longitudinal bars (indexed by i) engaged by a stirrup corner or a cross-tie.

The fixed-end-rotation of the end section due to slip of longitudinal bars from their anchorage increases between yielding of the end section and ultimate cyclic curvature there as:

$$\Delta\theta_{slip,u-y} = 5.5 d_{bL}\varphi_u \tag{11}$$

For φ_u, φ_y and $\Delta\theta_{slip,u-y}$ as above, the plastic hinge length L_{pl} is [Biskinis and Fardis, 2008]:

- in beams, rectangular columns or walls, members of T-, H-, U- or hollow rectangular section in cyclic loading: $$L_{pl} = 0.2h + L_s/15 \tag{12a}$$
- for circular columns or piers with diameter D: $$L_{pl} = 0.6D + 0.09L_s \tag{12b}$$

If $\theta_{u,m}$ is calculated as above with the help of Eqs. (7)-(12), the safety factor for its conversion to a characteristic value via Eq. (6) is γ_{Rd} =2.

In beams, rectangular columns or walls and members of T-, H-, U- or hollow rectangular section $\theta_{u,m}$ (in rads) may also be estimated by a purely empirical expression, with potentially wider scope and smaller model uncertainty, giving a lower safety factor for Eq. (6), γ_{Rd}=1.75:

$$\theta_{u,m}^{pl} = \theta_y + 0.017\left(1-0.05\max\left(1.5;\ \min\left(10;\ \frac{h}{b_w}\right)\right)\right)(0.2^{\nu})\left(\frac{\max(0.01;\ \omega_2)}{\max(0.01;\ \omega_1)}\frac{L_s}{h}\right)^{\frac{1}{3}} f_c^{\,0.2}\, 25^{\left(\frac{\alpha\rho_w f_{yw}}{f_c}\right)} 1.225^{100\rho_d} \tag{13}$$

where [Biskinis and Fardis, 2008]:

- θ_y is given from Eqs. (2a), (2b);
- $\nu=N/bhf_c$, with b = width of compression zone, N = axial force (positive for compression);
- $\omega_1=(\rho_1 f_{y1}+\rho_v f_{yv})/f_c$: mechanical reinforcement ratio for the entire tension zone, including the tension chord (index 1) and the web longitudinal bars (index v);
- $\omega_2=\rho_2 f_{y2}/f_c$: mechanical reinforcement ratio for the compression zone;
- $L_s/h=M/Vh$: shear-span-to-depth ratio at the section of maximum moment;
- α: confinement effectiveness factor from Eq. (10a);
- $\rho_w=A_{sh}/b_w s_h$: ratio of transverse steel parallel to the plane of bending;
- ρ_d: steel ratio of diagonal bars (if any) in each diagonal direction of the member;
- b_w: width of one web, even in cross-sections with two or more parallel webs.

If Eq.(5) is violated at a member end, means to meet it are:

1. to increase the confining reinforcement ratio (i.e., ρ_{sx} at the exponent of the 2nd term before the last one in Eq. (13)) over an appropriate length near the end in question; this may be the only practical means to increase the chord rotation capacity of circular piers; all other measures below are limited to sections with one or more rectangular parts;
2. in a beam, to increase its bottom reinforcement (see term ω_2 in Eq. (13)); and/or
3. to replace part of any "web" reinforcement distributed between the tension and the compression one with a smaller total amount of tension plus compression reinforcement, to increase ω_2 in Eq. (13) and reduce ω_1 (which is the sum of the tension and "web" reinforcement) while keeping M_y and $(EI)_{eff}$, unchanged; and/or
4. in a column which is squat in a single plane of bending or in a short beam, to add diagonal bars (preferably replacing longitudinal ones, to avoid increasing M_y and $(EI)_{eff}$); and/or
5. in members where Eq. (13) applies, to increase the width of narrow webs, b_w (cf. term involving h/b_w); in a rectangular section this will also increase the width of the compression zone, b, and reduce the axial load ratio, ν, which is normalized by bh.

Measures 2 and 5 increase M_y and $(EI)_{eff}$. If the increase is small and limited to few members, it may not be worth revisiting any previous steps. The same applies for any measure where involving the longitudinal reinforcement, if care is taken not to increase M_y or $(EI)_{eff}$. Such increases are anyway safe-sided, as they reduce the seismic chord rotation demands in Step 9.

Step 11 – Capacity Design of force-controlled mechanisms and of sensitive components:

- The shear force demands, V_{CD}, in all members and joints,
- the seismic internal forces and deformations in components of multi-pier bridges which are to remain elastic (i.e., in the deck, fixed bearings, shock transmission units, seismic links consisting of shear keys, buffers and/or linkage bolts or cables, etc.),
- the seismic internal forces in the foundation system, and
- the forces transferred to the ground,

are estimated assuming that the intended or likely plastic hinges develop the design value of their moment resistance times an overstrength factor, γ_o, greater than 1.0 and as high as 1.35 for strain-hardening, and using the final longitudinal reinforcement in all relevant members ("Capacity Design"). In frame beams or columns, in bridge piers and in joints between a horizontal member (a beam – including foundation ones – a bridge deck, etc.) and a vertical one (frame column, wall, bridge pier, etc), this calculation may be based on equilibrium alone. Equilibrium is not sufficient for the calculation of capacity design forces in walls of buildings, at the interface between foundation elements and the ground, or in the deck and those components of multi-pier bridges which are designed to remain elastic after plastic hinging in the piers. In such cases capacity-design forces may be found assuming that the seismic action effects at the instant the moment capacities at plastic hinge are reached are proportional to the corresponding outcomes of the elastic analysis from Step 9.

Members or joints are dimensioned so that the design value of their shear resistance exceeds their capacity-design shear force. For members, critical is the check of shear resistance in the plastic hinge, $V_{Rd,cyc}(\mu_\theta)$, which degrades with increasing chord rotation ductility factor at the corresponding end, μ_θ:

$$V_{G+\psi Q+P} + V_{CD} \leq V_{Rd,cyc}(\mu_\theta)/\gamma_{Rd} \quad (14)$$

$V_{G+\psi Q+P}$ in Eq. (14) is the shear force in the member due to the concurrent quasi-permanent loads G+ψQ+P and μ_θ is the value of θ_{Ed} at the member end from Step 9 divided by the chord rotation at yielding there θ_y, as this comes out of Eqs. (2) for the member final longitudinal reinforcement and the shear span L_s (moment-to-shear ratio) from the analysis in Step 9.

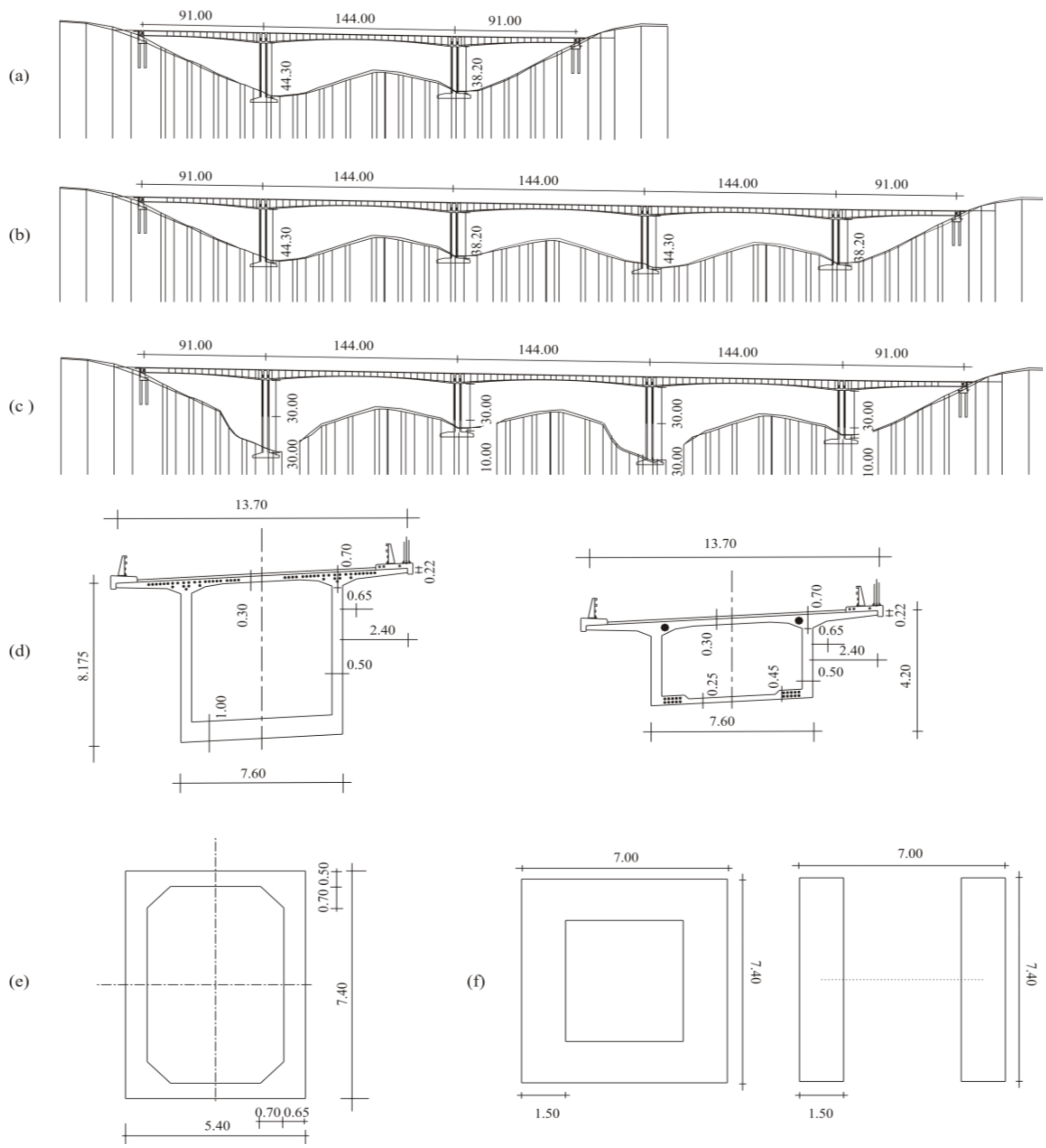

Fig. 1: Type-C3 bridges: (a) C3; (b) C3a;(c) C3b; (d) deck sections near supports to piers (left) or at midspan (right); (e) pier section of bridges C3 and C3a; (f) pier section of bridge C3b: lower hollow part (left) upper twin-blade part (right)

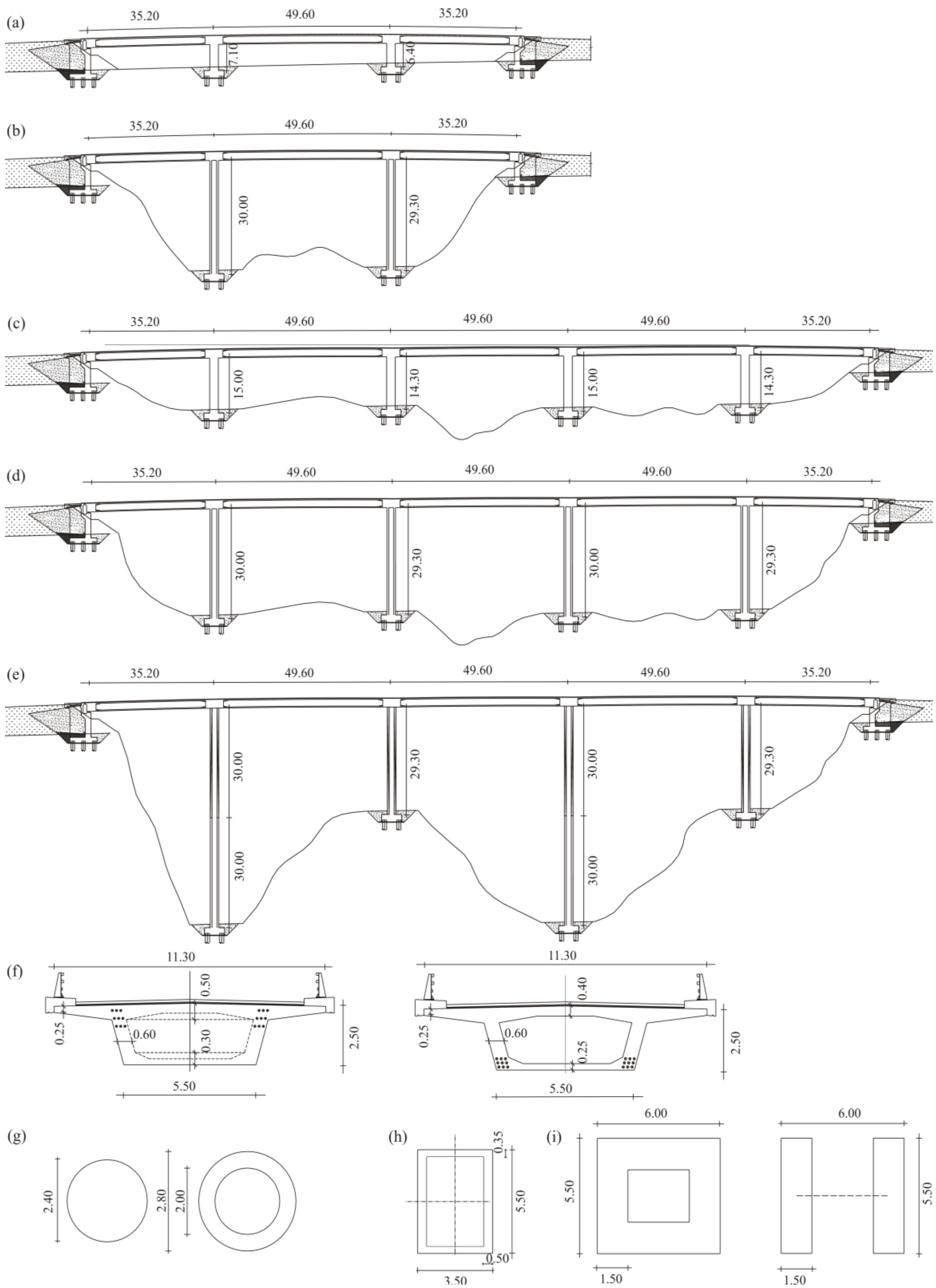

Fig. 2: Type-T6 bridges: (a) T6; (b) T6a;(c) T6b; (d) T6c; (e) T6d; (f) deck sections near supports to piers (left) or at midspan (right); (g) pier section in bridges T6 and T6b (left) or T6a (right); (h) pier section of bridge T6c; (i) pier section of bridge T6d: lower hollow part (left) upper twin-blade part (right)

The entire foundation system and the ground, as well as the deck and other components of multi-pier bridges meant to remain elastic, are also verified at the ULS for their force demands from capacity design. Eq. (14) is also symbolic of the force-based verification of all these elements, with the 1st term standing for internal forces due to the concurrent quasi-permanent loads and the 2nd one for the seismic action effects from capacity design. At the right-hand-side of the generalized verification is the design value of the corresponding force resistance (with design values of material properties), further divided by a model uncertainty factor. If any dimensions turn out to be insufficient, Steps 2 to 10 are repeated as necessary.

Step 12 - Evaluation of the design via nonlinear dynamic analysis: For regular structures (including bridges with piers of similar height) this step is optional. It is essential for irregular ones, as 5%-damped elastic response spectrum analysis may underestimate in Step 9 the inelastic deformation demands.

EXAMPLE APPLICATION TO EIGHT BRIDGES AND CONCLUSIONS

The outcome of the application of the procedure is demonstrated here for 8 bridges. They have prestressed box-girder deck, free longitudinally but fully restrained transversely at the abutments, integral with piers of similar or different heights and circular, annular, hollow rectangular or twin-blade section. They are variants of two real bridges, C3 and T6 (Figures 1 and 2), giving a set of 8 different bridges with 3 or 5 spans. To harmonize the longitudinal stiffness of the piers in bridges C3b and T6d, the top 30m of their piers consist of “twin blades” with the long dimension in the transverse direction of the bridge; the lower part of the pier has a stiff box section (Figures 1(c), (f), 2(e), (i)). The Life Safety earthquake has peak ground acceleration (PGA) of 0.21g for the type-C3 bridges or 0.16g for the type-T6 ones.

Each bridge has been designed either with: (a) the conventional, force-based seismic design of Eurocode 8, or (b) the proposed seismic design procedure and just the prescriptive detailing rules for non-earthquake resistant bridges [Bardakis and Fardis, 2008]. To facilitate the comparison, the two versions of each bridge have exactly the same deck and the same pier section. Only the pier reinforcement changes between the two designs and indeed comes out of the proposed procedure much lighter (see Tables 1 and 2). However, this is not at the expense of the seismic performance, as exemplified in Table 1. Detailed results [Bardakis and Fardis, 2008] show that the proposed procedure gives a more balanced design, with less inelasticity in the deck, more uniform deformation demands among the piers and damage in shear or flexure which is not more severe than in the Eurocode 8 designs.

Table 1. Peak Ground Acceleration (PGA) at which safety margin of 2.0 on flexural capacity or of 1.25 on shear resistance is exhausted anywhere in the bridge and total amount of steel in the piers

Bridge	Eurocode 8		Proposed design	
	PGA	steel in piers (t)	PGA	steel in piers (t)
C3	0.45g	219	0.57g	153
C3a	0.81g	437	0.80g	306
C3b	0.73g	1014	0.73g	897
T6	0.49g	15	0.45g	5
T6a	0.49g	48	0.45g	25
T6b	0.63g	36	0.60g	13
T6c	0.64g	158	0.63g	84
T6d	0.55g	512	0.56g	343

Table 2. Pier reinforcement ratios (%): vertical ρ_L, horizontal/confining: $\rho_{w,L}$ in bridge's longitudinal direction, $\rho_{w,T}$ in transverse (denoted by ρ_w if $\rho_{w,L}=\rho_{w,T}$)

bridge	location in pier and type of reinforcement		Eurocode 8		Present design							
			solid or box piers	30m-tall twin blades	solid or box piers		30m-tall twin blades in:					
							outer long pier		central long pier		short piers	
					long	short	top half	bottom half	Top half	bottom half	top half	bottom half
C3 and C3a	full height	ρ_L	1.05	-	0.76	0.48	-	-	-	-	-	-
	in plastic hinges	$\rho_{w,L}$	0.67	-	0.41	0.36	-	-	-	-	-	-
		$\rho_{w,T}$	0.88	-	0.51	0.44	-	-	-	-	-	-
	outside plastic hinges	$\rho_{w,L}$	0.34	-	0.37	0.29	-	-	-	-	-	-
		$\rho_{w,T}$	0.44	-	0.46	0.36	-	-	-	-	-	-
C3b	full height	ρ_L	1.61	1.00	1.61		0.68	0.39	0.68	0.39	0.68	0.39
	in plastic hinges	$\rho_{w,L}$	1.57	0.48	1.57		0.48	0.42	0.48	0.42	0.48	0.42
		$\rho_{w,T}$	1.58	0.46	1.58		0.46	0.40	0.46	0.40	0.46	0.40
	outside plastic hinges	$\rho_{w,L}$	0.88	0.24	0.88		0.24	0.21	0.24	0.21	0.24	0.21
		$\rho_{w,T}$	0.89	0.23	0.89		0.23	0.20	0.23	0.20	0.23	0.20
T6	full height	ρ_L	1.56	-	0.24	0.21	-		-	-	-	-
	top half	ρ_w	0.55	-	0.29	0.32	-		-	-	-	-
	bottom half	ρ_w	0.55	-	0.37	0.43	-		-	-	-	-
T6a	full height	ρ_L	1.00	-	0.51		-	-	-	-	-	-
	in plastic hinges	ρ_w	1.57	-	0.81		-	-	-	-	-	-
	outside pl. hinge	ρ_w	0.79	-	0.41		-	-	-	-	-	-
T6b	full height	ρ_L	1.00	-	0.21		-	-	-	-	-	-
	in plastic hinges	ρ_w	0.40	-	0.27		-	-	-	-	-	-
	outside pl. hinge	ρ_w	0.20	-	0.14		-	-	-	-	-	-
T6c	full height	ρ_L	1.00	-	0.51		-	-	-	-	-	-
	in plastic hinges	$\rho_{w,L}$	0.57	-	0.29		-	-	-	-	-	-
		$\rho_{w,T}$	0.57	-	0.36		-	-	-	-	-	-
	outside plastic hinges	$\rho_{w,L}$	0.51	-	0.26		-	-	-	-	-	-
		$\rho_{w,T}$	0.51	-	0.32		-	-	-	-	-	-
T6d	full height	ρ_L	1.00	1.05	1.00		0.88	0.26	0.54	0.21	0.41	0.26
	in plastic hinges	$\rho_{w,L}$	0.37	0.70	0.37		0.50	0.27	0.35	0.30	0.35	0.27
		$\rho_{w,T}$	0.35	0.68	0.35		0.48	0.26	0.33	0.28	0.33	0.28
	outside plastic hinges	$\rho_{w,L}$	0.19	0.34	0.19		0.25	0.14	0.18	0.15	0.18	0.13
		$\rho_{w,T}$	0.18	0.33	0.18		0.24	0.13	0.17	0.14	0.17	0.14

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