CURRENT TRENDS IN EUROPEAN EARTHQUAKE RESISTANT ANALYSIS AND DESIGN OF REINF. CONCRETE STRUCTURES

AVRUPA'DA BETONARME YAPILARIN DEPREME DAYANIKLI ANALİZ VE TASARIMINA İLİŞKİN GÜNCEL EĞİLİMLER

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ABSTRACT

The recent Eurocode 8 is considered to represent the latest word in codified earthquake resistant design. The implications of its provisions for reinforced concrete structures are presented. It is shown, through the results of its application to several typical reinforced concrete buildings, that the three ductility classes provided by the code are almost equivalent, in terms of materials cost and of achieved performance. Particular emphasis is given to the impact of the code provisions on the analysis and design process, which becomes relatively complex and computationally demanding. Practical ways of expediting this process are suggested. Possible pitfalls from the blind application of the code provisions are indicated and points where the latter may be improved in the future are suggested. In a different theme, the basic idea of the recently proposed displacement-based design concept is presented, and the differences from the conventional force/strength-based approach are identified. A general scheme for the application of the concept to multistory reinforced concrete buildings is proposed.

INTRODUCTION: THE EUROCODES

One by one the Eurocodes have become in the last few years prenorms (ENV) in the countries of CEN (Comité Européen de Normalisation). The set of Eurocodes covers in a unified and internally consistent way the design actions (permanent and occupancy loads, wind, snow and seismic actions, etc.), rules for the design and construction of concrete, steel, composite (steel-concrete), masonry and timber structures, foundations, retaining structures and geotechnical aspects and, last but not least, earthquake resistant design. It includes separate parts for buildings, bridges, towers, silos, tanks, masts, chimneys and pipelines, and is supplemented with other European Norms or ENVs on materials, such as ENV206 for concrete. Regarding concrete, EC2 covers not only the most common case of cast-in-situ normal weight reinforced or prestressed concrete, but also precast, lightweight, or plain (unreinforced) concrete and prestressing with unbonded tendons.

During their tenure as prenorms, the Eurocodes are undergoing testing through trial applications and will be the subject of formal inquiries within the CEN countries,

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in order to become more operational and to narrow the gaps with the diverse and often conflicting national standards currently in use. At the end of this period and around the year 2000 the Eurocodes are expected to be revised and converted into European Norms (ENs). Formally this will mean that CEN countries will have to withdraw national standards conflicting with them and to give the Eurocodes the status of national standards, with some allowance, though, for limited variations ("boxed values", "National application documents") to cope with deep-rooted differences in local design and construction traditions. Due to its completeness, rationality and internal consistency and its future application as the single structural design standard in the unified market of the continuously growing group of EU and affiliated countries, the Eurocodes seem bound to penetrate the international market and to strongly affect national standards even in countries far away from Europe.

EUROCODE 8 PARTS 1-1 & 1-2: SEISMIC ACTIONS, GENERAL REQUIREMENTS FOR STRUCTURES AND GENERAL RULES FOR BUILDINGS.

Part 1-1 of the recently finalised and approved as ENV 1998-1-1 to 1-3, Eurocode 8: "Earthquake Resistant Design of Structures, Part 1-1: General Rules and Rules for Buildings", contains mainly the general requirements and the definition of the seismic action. The requirements are two: Structures are required to withstand the design action without local or general collapse and to suffer little structural or non-structural damage under a "service" earthquake of moderate intensity with higher probability of exceedance than the design ground motion. The design seismic action is generally given in terms of a design ground acceleration with a mean return period of 475 yrs (10% exceedance probability in 50 yrs) for usual structures and a normalised elastic acceleration response spectrum with a soil-type-dependent standard shape. As the code adopts the conventional force/strength-based design approach outlined at the begining of the last Chapter of this paper, design is based on a design acceleration response spectrum, generally derived by dividing the ordinates of the elastic spectrum by the "behavior" factor q. To take into account in a conservative way dynamic effects on flexible structures, in the constant velocity and the constant displacement ranges elastic spectral ordinates are magnified by a factor proportional to $T^{1/3}$. The design spectrum includes the novelty of a period-dependent q-factor for the low-period rising range of the elastic spectrum, such that at T=0 the design acceleration is always equal to the ground acceleration, changing thereafter linearly up to the constant acceleration plateau.

Part 1-2 deals with buildings and sets the general rules for their structural layout (regularity in plan and in elevation), for the methods and the loads and load combinations to be used in the analysis and for the verification of the ULS (Ultimate Limit State - no collapse requirement) and the SLS (Serviceability Limit State - limitation of damage). Regarding regularity, structures which are classified as irregular in plan (either because their shape in plan deviates from the rectangular by more than 25% in one of the two directions, or because at any story they develop under the equivalent lateral seismic loads acting at a 5% eccentricity with respect to the center of mass, maximum horizontal drifts exceeding the mean story drift by more than 20%), should be analyzed in 3D. However, as we will see below, this "penalty" has little practical significance.

Structures which are classified as irregular in elevation, either because they have story-to-story setbacks exceeding 10% on one side and 30% total over the height if on one side (single setbacks of less than 50% in the lower 15% of the building are not considered as source of irregularity), or because one or more vertical elements do not run uninterrupted from top to bottom, are penalised by a 20% reduction in the q-factor (i.e. increase of the design action by 25%) and by the requirement to perform a full multimodal analysis instead of a static one for lateral loads proportional to a linearly increasing with height first mode shape.

The analysis is performed with gross (uncracked) section stiffnesses of RC members, and inertial forces are computed for simultaneous action of the quasi-permanent part (30% for residential and office buildings) of occupancy loads, reduced by half for all independently occupied stories other than the top. With the exception of structures which are classified as regular in plan and in elevation (for which the analysis can be static for the above mentioned "inverted-triangular" equivalent lateral loads and accidental torsion effects can be approximately accounted for by multiplying seismic internal forces by a factor which increases linearly from 1.0 at the center of mass of the plan to 1.3 at the outermost structural element), the multimodal response spectrum analysis should be applied, with complete quadratic combination of the contributions of modes with periods which differ by less than 10%, and SRSS combination for all other modes. The number of modes to be considered in each direction should provide a total effective modal mass of at least 90% of the real mass. The effects of a 5% accidental eccentricity of static inertial lateral forces in each horizontal direction should be added to the results of the multimodal dynamic analysis.

Each horizontal component of the earthquake is considered to act together with 30% of the horizontal component in the orthogonal direction. In addition, in structures with beams spanning over 20m, or prestressed, or supporting vertically interrupted columns, the vertical component of the earthquake should be considered also, combined with 30% of both horizontal components.

EC8 is characterised by a high degree of sophistication, rationality and technical completeness, allowing the design of structures which are at the same time economic and safe, even against an extremely uncertain action, such as the seismic. However this power is at the expense of simplicity, of the contents of the code itself and of its application. At the top of the list of the most computationally demanding code provisions are those regarding the combinations of the seismic action (simultaneous action of earthquake components, 5% accidental eccentricities of the masses in plan, etc., taken with all possible combinations of signs), the variety and sophistication of the methods of analysis, depending on the degree of irregularity of the structure, etc. Moreover, the capacity design rules of Part 1-3, Concrete, introduce considerable coupling between the various phases of design of the same element (design for bending and determination of longitudinal reinforcement in beams, columns and walls and design for shear) and the design of different elements (capacity design magnification of column moments depending on the flexural capacities of beams and their seismic moments from the analysis, verification of a beam-column joints in shear depending on the longitudinal reinforcement of the elements framing into it, etc.). Because of the amount and complexity of the computations required by Parts 1-1 and 1-2 and of the volume of analysis and design information to be stored and transferred for the needs of the various phases of design according to Part 1-3, use of a computer and of the appropriate software is essential for the application of EC8 for the design of even very simple structures.

With the above in mind, the criterion for the operationality of the code provisions is their generality and applicability to the entire spectrum of design situations. Therefore, supposing that the designer has at his disposal a general computational tool which allows analysis in 3D ("spatial"), it makes computationally no sense to use two separate 2D "plane" models to analyze structures which are regular in plan, as allowed by Part 1-2, and to combine then (manually?) the results of the two separate planar analyses and those for the gravity loads (in 2D or 3D?) for the purposes of proportioning and verification. Similarly, as the multimodal response spectrum ("dynamic") analysis not only captures better the effects of irregularities in elevation on the dynamic response, but also gives overall about 10% more economic (less conservative) results for regular structures than the equivalent static, it makes sense to use the former instead of the latter as the standard method of analysis even for regular structures. The implication for member proportioning is that, since the SRSS and the "complete quadratic combination" rules for the combination of modal contributions to each internal force give positive results, the latter have to be considered in the various combinations of actions with a + and a - sign. Finally, accounting for torsional effects through static analysis with torsional story moments resulting from static story horizontal forces according to the general provisions of Part 1-2, should be preferred over the approximate and less general alternatives allowed therein or in Annex B of Part 1-2 for regular in plan structures, as it gives more accurate and economic results for proportioning.

For the purposes of member proportioning it is not necessary to compute all $2^5 = 32$ values of an internal force resulting from the combinations of signs of the two horizontal components of the seismic action with the signs of the corresponding torsional effects, etc. Two values, one for each horizontal direction E_x and E_y of the seismic action, each with a + and a - sign, are enough instead: the absolute value of the internal force due to E_x (resp. E_y) is added to that of the corresponding internal force due to the accidental eccentricity of E_x (resp. E_y), e_{Ex} (resp. e_{Ey}); the result of this combination of E_x and e_{Ex} (resp. E_y and e_{Ey}) is added to 30% of the corresponding combination of E_y and e_{Ey} (resp. E_x and e_{Ex}), according to par.3.3.5.1(3) of Part 1.2. The resulting two positive values are taken then in proportioning with + and - sign.

EUROCODE 8 PART 1-3, CONCRETE: BEHAVIOR FACTORS AND DUCTILITY CLASSES

The concrete part of EC8 Part 1-3 gives the values of the behavior factor q, for RC structures, as a reference value which depends on the type of the structural system reduced for special effects such as irregularity in elevation, squat shear walls and local and overall ductility. If the minimum for all horizontal directions torsional radius of the structure (square root of the ratio of the torsional to the translational stiffness of the vertical elements) is at least 80% of the radius of gyration of its plan, the reference value of the behavior factor, q_o, is 5.0 if frames or coupled walls take at least 50% of the base shear, and 4.5 or 4.0 if non-coupled walls take between 50% and 65% or over 65%, respectively, of the base shear ("wall-equivalent" and "wall" systems). Systems not satisfying the minimum torsional rigidity condition above ("core" systems) have q_o equal to 3.5, while in those with at least 50% of the mass in the upper third of the height or with most of their resistance concentrated at the base of a single element ("inverted pendulum"), q_o is equal to 2.0. In "wall-equivalent", "wall" and "core" systems with average wall aspect ratio, $\alpha_o = \Sigma h_w / \Sigma l_w$, less than 3, the q factor is reduced by dividing q_o by $2.5\text{-}0.5\alpha_o$, while an irregularity in elevation according to Part 1-2 is penalized by a reduction in the q value by 20%.

Part 1-3 provides for three "Ductility Classes", DCH (High), DCM (Medium) and DCL (Low), according to the energy dissipation capacity of the structure and of its members, as this capacity is determined from the prescribed detailing and capacity design rules of this Part of EC8. In DCM and DCL the q factor is reduced by 25% and 50% respectively over the reference value after any reduction for irregularity in elevation or wall aspect ratio. Therefore for the same ground acceleration a DCL structure is designed for 1.5 the base shear of a DCM one, which in turn is designed for 1.33 the base shear of a DCH structure. On the other hand, the capacity design and detailing requirements of Part 1.3 become more demanding with increasing ductility class. The idea is to leave to the member countries the selection of the ductility class(es) for which structures in the various countries will be designed. DCL will be more appropriate for low seismicity regions with little emphasis and tradition in earthquake resistant design and construction. DCH, on the other hand, may be best for high seismicity regions with high standards and tradition in workmanship and quality control, especially as far as placement of the reinforcement is concerned.

The writer has designed 22 RC frame and 4 "core" wall buildings in 3D, ranging from 3 to 12 stories, to study the effect of designing the same building for a different ductility class. It was found that in frames the three DCs lead to about the same total quantities of steel and concrete, with a shift in the ratio of column-to-beam total steel from 55%-45% for DCL, to 60%-40% for DCM and 65%-35% for DCH, and a change in the ratio of longitudinal-to-transverse steel from 80%-20% for DCL, to 75%-25% for DCM and 60%-40% for DCH. In the wall structures, irrespective of DC this ratio came out around 55%-45%, while the fraction of the total steel in the vertical elements is about equal to the fraction of the base shear taken by the walls.

The shift of the quantity of steel in frames from beams to columns with increasing ductility class is due on the one hand to the fact that proportioning of the beam longitudinal reinforcement is based on bending moments from the analysis, which decrease by 25% when we go from DCL to DCM and by one third when we go from DCM to DCH, and on the other to the capacity design magnification of column moments in DCM and DCH, which increases design moments over those resulting from the analysis for DCL. The shift from longitudinal steel to transverse is due on the one hand to differences in the design for shear (capacity design determination of design shear force in beams of DCH and in columns of DCH and DCM, reduction in the contribution of concrete to shear strength with increasing ductility class, etc.) and on the other to the more stringent minimal measures on stirrups of beams and columns and to the more demanding confinement provisions for columns of higher ductility class.

The 22 frame structures above are been subjected to nonlinear dynamic analyses for 4 input motions compatible to the elastic response spectrum of EC8 and scaled to 1.0, 1.5 and 2.0 times the ground motion intensity. Preliminary results show very satisfactory performance (very little structural damage) even under twice the design ground motion, and very similar fragility curves (i.e. increase of total and interstory drifts and structural damage with ground motion intensity) for the different ductility classes. For the same design ground acceleration, designing for higher DC results in only slightly better nonlinear behavior, in terms of plastic hinging and member damage. This confirms the equivalence of the three ductility classes in terms of performance and reliability. In view though of the minor and non-systematic effect of the DC on the total quantities of steel and concrete, these results may raise doubts over the adviceability of applying a class such as DCH, which is very demanding in workmanship and design.

EC8 PART 1-3, CONCRETE: MEMBER PROPORTIONING AND DETAILING

Despite the apparent volume of the R.C. part of EC8 Part 1-3, the writer has summarized in 4 pages of Tables practically all EC8 and EC2 provisions for proportioning and detailing of beams, columns and walls (Fardis 1994). The information in those Tables may be sufficient to a designer familiar with the basic rules and symbols of EC2 and the principles of modern seismic design. Some comments on the implications of the proportioning and detailing provisions are given below.

Exceedance of the maximum steel ratio, ρ_{max}, at the top flange of the supports is usually the single limiting factor for the cross-sectional dimensions of DCH and DCM beams. Due to the dependence of ρ_{max} on ρ'/ρ, slightly undersizing the top reinforcement and oversizing the bottom one assists in observing ρ_{max}. The limitation on the maximum diameter of beam bars crossing beam-column joints often controls the cross-sectional dimensions of columns, esp. in low-rise buildings or in corner columns of high-rise ones where the minimum over all seismic load combinations axial force is low.

Regarding columns, if the analysis for the seismic action is "equivalent" static, as allowed for structures which are regular both in plan and in elevation, the signs of internal forces are maintained and column cross-sections can be verified for only 4 seismic action M_x-M_y-N triads. For multimodal response spectrum analysis, the need to consider all combinations of signs of M_x, M_y, N leads to $2^3=8$ M_x-M_y-N triads for each horizontal component of the seismic action, i.e. 16 total. Biaxial verification for these triads is more general and accurate and gives more economic results than the uniaxial $M_x/0.7$-N, $M_y/0.7$-N simplification allowed for columns of DCM and DCL structures.

The novel capacity-design magnification of column moments from the analysis, is very general and hence computationally convenient. The trial designs of the 26 buildings have shown that it gives results which are reasonable, economic and satisfactory from the point of view of column safety.

Columns stirrups are usually dictated by the minimal measures on their diameter and spacing. The requirement on stirrup volumetric ratio for confinement is seldom controlling, and if so only at first story corner columns where seismic axial load is maximum.

In contrast to those for beams and columns, the provisions for DCH and DCM walls seem to have limitations. They are too complicated in view of our imperfect knowledge of the seismic behavior of walls. Moreover, they cover practically only rectangular single walls and their application to coupled and/or channel- or L-shaped walls, either is not obvious or leads to practical problems. For example, in coupled walls use of the bending moment of the individual walls may lead to unreasonably high capacity design magnification factors for shear and to too low shear-spans in the shear-span-ratio-dependent dimensioning of the web reinforcement. Also, for non-rectangular sections the information given in the code is not enough for the calculation of hoops in the boundary elements. Last but not least, simultaneous application of certain empirical design rules may lead to very conservative designs. An example of such rules, which seem to be different means to achieve the same goal, is the vertical shift rule of the linear bending moment envelope of walls, and the empirical rules for the calculation of vertical web reinforcement for low shear span ratios in slender walls.

DISPLACEMENT-BASED DESIGN OF RC BUILDINGS

Despite the intense international activity of recent years and the immense progress of seismic design codes, it is generally recognised that seismic performance is not closely related to codified earthquake resistant design (e.g. Priestley 1993, Moehle 1992). For reasons of tradition and convenience we are still designing our structures for specified internal forces, while what determines seismic behavior and safety is the magnitude and the distribution of deformations. Seismic design is still based on elastic acceleration response spectra, from which the maximum base shear of an equivalent elastic structure is computed, along with the corresponding lateral loads at floor levels. These loads, which are considered to correspond to the peak inertial forces on the structure, are determined by dividing the corresponding forces in the elastic structure by the "behavior" factor q. Therefore what determines the magnitude of these loads is the elastic characteristics of the structure (e.g. the elastic stiffness), which have little relation with its actual seismic behavior. Members are then proportioned for the internal forces computed from an elastic analysis for these lateral loads. Therefore, despite the fact that under the design earthquake members may fail due to the magnitude of induced inelastic deformations, they are proportioned for resistance against specified forces (instead of against specified deformations), resistance which will certainly be exhausted by seismic events of significantly lower magnitude and higher exceedance probability than the design earthquake. The value of the behavior factor is constant throughout the structure and is considered to reflect on one hand the magnitude of the inelastic deformations that its members can resist, and on the other the global behavior and safety margins of the building, depending on its structural type and global configuration (regularity, symmetry, indeterminacy, etc.). In addition capacity design provisions are applied, which aim at imposing a desired mode of behavior and at reducing the uncertainty of the response to modelling and analysis assumptions. With this additional safety valve, seismic design is considered to lead to safe and conservative results. However, this final outcome and the semi-empirical way in which it is achieved, justify neither the sophistication of the preceding analysis (often dynamic in three dimensions), nor the effort required for the proportioning and detailing of members. Last but not least, the calculation of peak displacements and deformations on the basis of the elastic analysis results errs on the unsafe (unconservative) side (Pristley 1993).

The reasons above make necessary the long-term revision of seismic design codes so that design is directly related to the control of the magnitude and the distribution of inelastic deformations in the structure under the design earthquake, and of the non structural parts under the serviceability ground motion.

Moehle (1992) proposed a general outline of a design procedure based on interstory drifts and on displacement instead of acceleration response spectra, using as a starting point the elastic stiffness and period and the strength of the structure. This procedure differs from the conventional one only because it requires a direct check of displacements rather than an indirect of ductilities. Priestley (1993) and Kowalsky et al (1995) proposed an alternative displacement-based design procedure for SDOF systems, according to which the designer selects the value of the ultimate displacement on the basis of the applied detailing, postulates then a reasonable value for the yield displacement so that the displacement ductility factor can be estimated, and then enters a damping-dependent set of elastic displacement response spectra to determine the effective nonlinear period which is consistent with the selected ultimate displacement and the value of damping that corresponds to the displacement ductility factor. From this period, the secant stiffness at ultimate displacement is computed and used, along with the value of the latter, to determine the required strength to be used in proportioning. Calvi and Kingsley (1995) generalised this procedure to MDOF bridges consisting of a flexible deck on several piers, by converting the MDOF system into a SDOF one, on the basis of a postulated deformed shape of the deck and the piers at ultimate conditions. The procedure is iterative, but for regular systems it converges very fast.

The above displacement-based design approaches have as final outcome internal forces to be used for proportioning. In that respect they do not differ from the force/strength design approach. Wallace (1992, 1995a, 1995b) has made one more step in the direction of a more genuine displacement/deformation based design of shear wall structures. Specifically, he determines the confining steel of the wall boundary elements from the strain distribution over the wall section determined at ultimate curvature conditions on the basis of equilibrium, the Bernoulli assumption and the amount and distribution of the vertical steel. The ultimate curvature demand in the wall plastic hinge region is computed from the wall ultimate top displacement demand, as determined from a 5%-damped elastic displacement spectrum using the cracked section stiffness and the "equal-displacement-rule" (i.e. the approximate equality of the displacements of the elastic and the inelastic system within the constant velocity range of the spectrum).

Along the line of the above worldwide efforts and in the direction of developing a design procedure in which proportioning of members will be based directly on peak deformation demands, the following step-by-step procedure is proposed herein for displacement/deformation based design of multistory reinforced concrete buildings:

1. The longitudinal reinforcement of beams and of the bottom cross-section of the vertical members (columns and walls) of the bottom story are proportioned on the basis of the ULS for factored gravity loads and of the SLS for a moderate intensity "service" earthquake (specified as a fraction of the design seismic action), whichever is more critical. The SLS proportioning will be based on the requirement for no yielding of the longitudinal steel and for linear elastic concrete in compression, and will use the internal forces computed from a linear static analysis of the structure in 3D, for an inverted-triangular distribution of lateral seismic loads. As the results of this analysis may be used for the estimation of the fundamental period in each one of the two horizontal directions (see 4. below), cracked section stiffnesses, representative of the member secant stiffness at yielding, should be used.

2. Proportioning of column vertical reinforcement above the base of the bottom story on the basis of capacity design at beam-column joints, with an overstrength factor on beam ultimate capacities significantly greater than 1.0.

3. (Preliminary) proportioning of transverse reinforcement in beams and columns as in step 1, or according to the capacity design rules for shear design of beams and columns, whichever is more critical.

4. Realistic estimation of the elastic fundamental periods of the structure in each one of the two horizontal directions, through the Rayleigh Quotient and on the basis of the lateral displacements and forces for the service earthquake from step 1. (Given that member longitudinal and transverse reinforcement is at least provisionally determined in steps 1-3 above, member secant stiffness at yield may be computed more accurately at this stage, e.g. by employing the Park and Ang (1985) procedure for the estimation of chord rotation at yielding, accounting also for shear cracking and deformations and for bar slippage in beam-column joints. These member stiffnesses may be used to verify the ones used in the analysis of step 1, or even to repeat it for more accurate T-values).

5. Using the values of the period from step 4, estimation of a structure peak drift demand in each of the two horizontal directions, from a 5%-damped elastic displacement spectrum with its constant velocity branch extrapolated in the constant acceleration range to compensate for the lack of validity of the "equal displacement rule" there. This displacement being essentially that of an equivalent SDOF system in each horizontal direction, it has to be converted into a top displacement, on the basis of an assumed distribution of story drifts (linearly increasing with height, consistent with the capacity-design calculations of step 2, or according to the elastic analysis results of step 1).

6. Final proportioning of the longitudinal and transverse reinforcement at beam ends and at the base of the bottom story columns and walls, so that at each one of these ends the ultimate chord rotation supply is equal to the drift ratio of the structure at the top (possibly times a model uncertainly factor γ_{Rd}). The fact that some interstory drift ratios will exceed the total drift ratio at the top is at least partly compensated by the interstory drift absorbed in chord rotations of columns and in shear distortions of beam-column joints. The final proportioning includes determination of the compression-to-tension steel ratio at beam ends, increases in beam widths to reduce the tension steel

ratio for given top reinforcement from step 1, determination of confining steel, etc.

Steps 4 and 5 can be replaced by a single one in which the peak drift demand in each direction is estimated iteratively at the same time as the effective nonlinear fundamental period from a damping-dependent set of elastic acceleration and displacement spectra, on the basis of the peak response acceleration of the structure (estimated as the ratio of the ultimate base shear to the effective mass) and of the damping ratio corresponding to the ratio of ultimate drift to drift at yielding. This, among others, requires estimation of the structure yield drift and ultimate base shear, the latter from either a static incremental nonlinear push-over analysis, or from a limit analysis for an assumed sidesway plastic mechanism. Given, though, the uncertainly regarding the relation between global damping and drift ductility factor, the added accuracy barely justifies the increased computational complexity.

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