Table 3. Corrosion rate of mild steel in industrial and sheltered area

Exposure area	Weight loss (g)	Corrosion rate (mmpy)	Duration (days)
Industrial	0.9327	0.116	24
Sheltered	0.1682	0.021	21

have been converted into $CaSO_4$, the presence of which was noticed in XRD. The SO_4^{2-} might have been formed from SO_2 (air pollutant) in the presence of oxygen. The latter is well supported by the observations of May and Lewis[15] on the decay of buildings due to the action of bacteria. The corrosion rate of mild steel exposed to polluted site was 0.116 mmpy, whereas that exposed to (sheltered area) unpolluted environment was only 0.021 mmpy. The increase in corrosion rate was five times that of control. It is claimed that the higher corrosion rate was due to the role of pollutant with air microbes on the corrosion of specimens and that the atmospheric corrosion of specimens exposed to polluted environment was due to significant contributions of combined action of HB, IB, MOB and thiobacilli.

Microorganisms enhance deterioration of materials of construction and steel structures. In the present study, *Pseudomonas*, *Bacillus*, *Micrococcus*, *Moraxella*, *Anthrobacter*, *Streptococcus*, *Staphylococcus* and *Acinetobacter* were identified as the air microbes on a concrete pile. It is concluded that the deterioration is due to the presence of bacteria. The study by FTIR indicated the presence of NH_2 group, C=O stretch and C–Cl, which was due to the adsorption of biofilms on the specimens exposed to the industrial atmosphere. $CaSO_4$ and FeS peaks were noticed on mild steel and concrete exposed to the industrial area. Increase in corrosion rate of mild steel was five times more compared to that of control. It is claimed that the higher corrosion rate of steel was due to the simultaneous action of pollutants with the air microbes on the specimens. The atmospheric corrosion of concrete and steel exposed to polluted atmosphere of the chemical industry might be due to Mn oxidizers, Fe oxidizers, HB and acid producers.

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A generalized method for seismic evaluation of existing buildings

Durgesh C. Rai

Department of Civil Engineering, Indian Institute of Technology, Kanpur 208 016, India

Current Indian codes do not address the evaluation of seismic resistance of existing building stock, which may not have been designed for earthquake forces. An appropriate level of safety needs to be ensured for occupants of these buildings through strengthening measures, if found deficient. Existing buildings not designed in accordance with the philosophies of current seismic codes need to be assessed for their expected seismic performance in future earthquakes. The proposed method provides for a consistent yet flexible framework to assess the ability of existing buildings to reach an adequate level of seismic performance related to life safety of occupants. The elements of

e-mail: dcrai@iitk.ac.in

assessment are arranged in a tier-like format for increasing level of detail and sophistication, which can be easily applied to a rather wide range of buildings, with mixed structural systems typically encountered in India. Further, the method can be easily extended or modified to include other structural systems as well.

Keywords: Buildings, earthquakes, life safety, seismic codes, seismic evaluation, vulnerability.

A GREAT many existing buildings are not designed for earthquake forces and their performance in future earthquakes is quite questionable, considering the performance of similar structures in the past. It is extremely important that an appropriate level of safety be ensured for the occupants. Current Indian safety codes do not address the evaluation of seismic resistance of existing building stock, so that proper remedial measures can be taken to reduce the risk of collapse or damage. Existing buildings not designed in accordance with the philosophies and requirements of current seismic codes, can be assessed using the procedures outlined in this communication. This paper also provides a method to assess the ability of an existing building to reach an adequate level of performance related to the life safety of its occupants. The systematic procedure outlined here can be used for seismic evaluation of buildings and can be applied consistently to a rather wide range of buildings.

In general, a classification of buildings and their deficiencies is necessary for seismic evaluation studies. However, such a classification is difficult, especially in building construction of old traditional societies like ours, where innumerable variations of structural configurations and use of materials are widespread. Even in the formal construction types, large variations do exist. The proposed guideline does recognize these variations and the difficulties that it poses in developing a consistent evaluation scheme. As a solution, the proposed methodology stresses on the lateral load carrying elements rather than on building type for evaluation of strength and ductility according to their structural action and material use. However, due care is given to building type in assessing seismic load and provisions for global stability, robustness, and ductility at the global and member level. Further, the framework is made such that certain peculiarities of specific building types can be easily entered through supplemental checks.

This proposed methodology is based on a prescriptive engineering approach which is in line with the current Indian practice. The emphasis is on identifying the most vulnerable elements of the buildings at risk, which alone can be strengthened with priority to reduce the risk of partial or complete collapse. The procedure presented has been derived from documents such as ATC 40 (ref. 1), FEMA 310 (ref. 2), FEMA 273 (ref. 3), (now FEMA 356)[4], UCBC[5] (now GSREB[6]) of ICBO, ASCE 31-03 (ref. 7), New Zealand Draft Code[8] and Eurocode 8 (ref. 9). In the following sections an overview of the proposed generalized method is presented; however, complete provisions and their commentary alongwith illustrated examples are available elsewhere[10]. This peer-reviewed guideline was discussed in a two-week long e-conference on Structural Engineering Forum of India (www.sefindia.org), where professionals from academia and industry expressed their views and comments. It was also been presented at two workshops held in Delhi and Ahmedabad for practising structural engineers. One of the objectives of this communication is to report the basic framework of the evaluation methodology so developed which can lead to further work and eventually result in a code from the Bureau of Indian Standards.

Seismic performance of existing buildings is evaluated in relation to the performance criteria in use for new buildings. The relationship between the design of a new building and evaluation of an existing building is depicted in Figure 1. The key concept lies in the fact that the loadings standard and materials standard are common to both of them; however, these standards are suitably modified to account for the present-day strength of materials and the remaining useful life. Minimum evaluation criteria are specified for the expected performance of life safety of existing buildings with appropriate modification to IS:1893 (ref. 11). All existing structural elements must

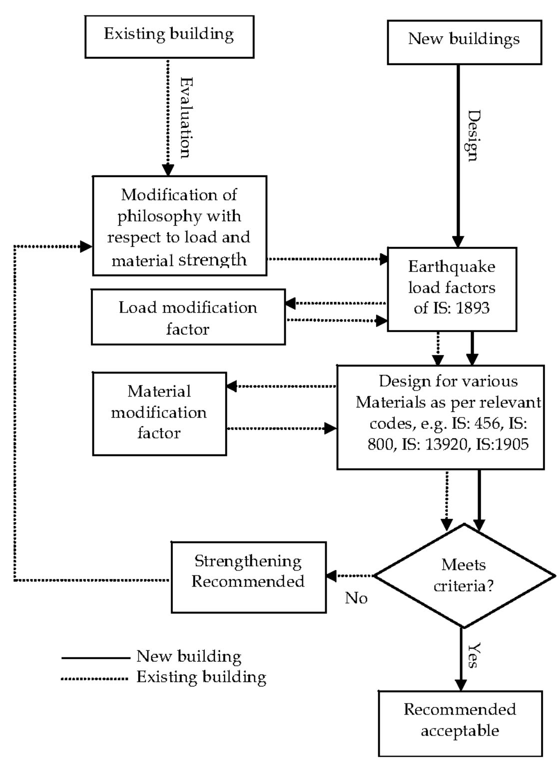

Figure 1. Relationship between procedure for design of new buildings and evaluation of existing buildings.

Table 1. Knowledge factor

Sl. no.	Description of building	*K*
1	Original construction documents available, including post-construction activities, such as modification to structure or materials testing undertaken of existing structure.	1.00
2	Documentation as above in (1) but no testing of materials, i.e. using originally specified values for materials.	0.90
3	Documentation as above in (1) but no testing of materials, i.e. originally specified values for materials and minor deterioration of original condition.	0.80
4	Incomplete but useable original construction documents and no testing.	0.70
5	Documentation as in (4) and limited inspection, and verification of structural members, or materials test results with large variation.	0.60
6	Little knowledge of details of a component.	0.50

be able to carry all other non-seismic loads in accordance with the current applicable codes related to loading and material strength.

Basic hazard parameters for determination of seismic forces are to be used directly from IS:1893; however, for important buildings a site-specific seismic design criteria can be specified. However, these forces need to be modified for the reduced useable life of the existing building. It is widely accepted to reduce the design force for evaluation purposes, which amounts to accepting a higher risk of loss for the existing buildings compared to new buildings, which finally results in lower costs for strengthening and upgradation. Various codes specify a reduction factor which is either constant or varies with the remaining useful life of the building and they are usually in the range 0.5–0.75. Reduced useable life factor U can be taken as 0.67 to modify the lateral force (base shear) for new building as specified in IS:1893.

Furthermore, the probable strengths of structural members need to be established based on structural drawings and present-day material strength. Quite often, accurate and reliable information is not available and requires detailed building investigation and field or laboratory tests to supplement the available information. The uncertainties associated with this information can be represented by a knowledge factor K, which is multiplied with estimated material strength. Table 1 provides some values for knowledge factor to be used depending upon the extent and reliability of information available from test or original building documents. These modifications to loading U, as well as material strength K, will be applicable throughout the evaluation process.

The seismic evaluation of existing buildings is primarily a two-tiered process which comprises of preliminary as well as detailed evaluation procedures. Preliminary evaluation is carried out first, which involves broad assessment of its physical condition, robustness, structural integrity and strength of structure, including simple calculations. A detailed evaluation is required when results of the preliminary evaluation are not acceptable. However, some minor buildings of single- or two-storeys with total floor area less than 300 sq. m, can be exempted from detailed evaluation. Seismic retrofitting is carried out to remedy the deficiencies identified during the preliminary process.

A detailed evaluation includes numerical checks on stability and integrity of the whole structure as well as the strength of each structural member. Conventional design calculations for these checks will use modified demands and strengths. Existing buildings which do not satisfy the acceptability criteria of either preliminary evaluation or detailed evaluation are recommended for strengthening of deficient members. The chosen seismic strengthening scheme should increase the redundancy of lateral load-resisting elements to avoid collapse and overall instability, and a detailed evaluation of strengthened building should yield satisfactory results. A flow diagram summarizing various steps of the evaluation process is shown in Figure 2.

The purpose of the preliminary evaluation process is to quickly identify the structural layout and building characteristics that determine its seismic vulnerability (Figure 3). It helps the design professional to become familiar with the building, its potential deficiencies and behaviour. It is an approximate procedure based on observed damage characteristics in previous earthquakes coupled with some back-of-the-envelope calculations. Further, the method uses conservative parameters to identify the potential earthquake risk of a building to screen them for detailed evaluation. Preliminary evaluation begins with a site visit and the building is said to be acceptable if it meets all the configuration-related checks as well as global level checks on axial and shear stress in lateral load-resisting members, as outlined below:

A site visit is conducted by the design professional as a first step to verify available existing building data or collect additional data, and to determine the present condition of the building and its components, which are usually not conveyed in drawings and other documents. Information with regard to its age, structural layout, lateral force resisting system, use and nature of occupancy, condition of materials and its components, architectural features and geotechnical details are obtained. The design professional should look for special anomalies and conditions, especially certain architectural features which may significantly affect the seismic performance.

Configuration-related checks are performed to verify that the building has all the essential lateral load-resisting features in good condition. The implementing provisions are the same as those described in IS:1893 and IS:4326 (ref. 12); and the general features of these are summarized in the following.

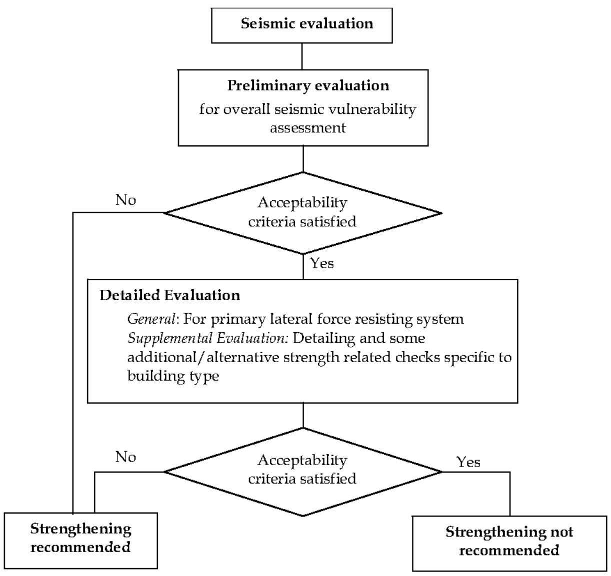

Figure 2. Seismic evaluation procedure for existing buildings.

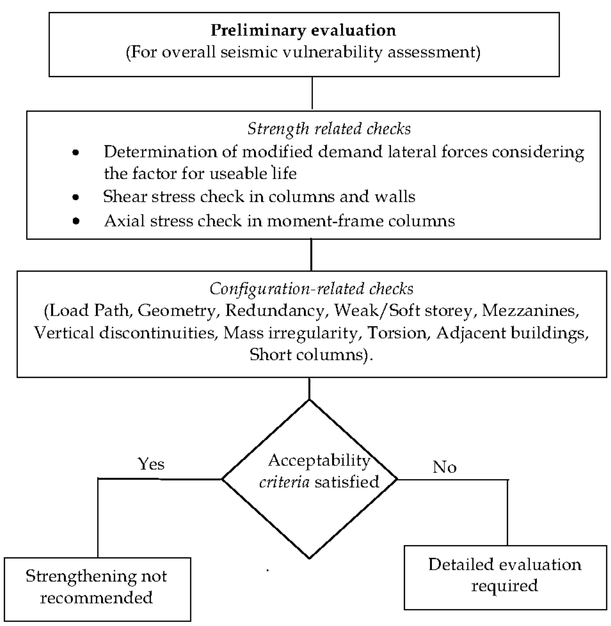

Figure 3. Various steps involved in preliminary evaluation.

Lateral load-resisting members should be tied together to act as a single unit for proper seismic response of a building during earthquakes. A vertical lateral force-resisting system should be continuous and should run from the foundation to the top of the building. The design professional should identify any gap in the load paths leading to incomplete transfer of lateral loads to the foundation.

Redundancy of seismic load path is necessary because of the uncertainties involved in the magnitude of both seismic loads and member capacities. If any member of a lateral force-resisting system fails, the redundancy of the structure will help ensure that there is another member present in the lateral force-resisting system that will contribute lateral resistance to the structure. Redundancy also provides multiple locations for potential yielding and possibly distributing inelastic activity at other locations within the structure, thus improving the ductility and energy dissipation capacity of the structure.

Geometric irregularity of overall building shape in plan and elevation affects the seismic response of the structure by increasing ductility demands at a few locations. Besides, an irregular shape indicates an irregular mass distribution, which may cause certain parts of the building to respond dynamically, independent of the rest of the building. It is recommended that the plan configuration be always symmetrical with respect to two orthogonal directions and it should be compact and represent simple shapes.

Mezzanines/lofts/sub-floors are often added on at a later date and lack lateral-force-resistant system and therefore pose a potential collapse hazard. When mezzanines are added onto the main structure, the supporting elements of the main structure should be evaluated considering both the magnitude and location of the additional forces imparted by the mezzanine. Also, the lateral-force-resisting elements should be present in both directions to act as a bracing.

Weak storeys are usually found where vertical discontinuities exist, or where member size or reinforcement has been reduced. The result of a weak storey is a concentration of inelastic activity that may result in partial or total collapse of the storey. Soft storeys are present in buildings with open fronts on the ground floor or tall ground storeys. Soft-storey buildings are well known for their poor performance during earthquakes. Both weak and soft storeys can be detected by respectively comparing the strength and stiffness of adjacent storeys. However, for detailed investigation, a nonlinear analysis will be necessary to determine if there are unexpectedly high seismic demands at locations of strength discontinuities.

Vertical discontinuities are usually found where elements are not continuous to the foundation but stop at some upper level. The most common example is a discontinuous shear wall or braced frame. The shear at this level could be transferred through the diaphragm to other resisting elements below. This force transfer can be accomplished either through a horizontal strut/tie if the elements are in the same plane or otherwise through a connecting diaphragm. While the strut or connecting diaphragm may be adequate to transfer the shear forces to adjacent elements, the columns which support vertical loads are the most critical. This irregularity results in a local strength and ductility problem below the discontinuous element, not a global storey strength or stiffness irregularity. Compliance can be achieved if an adequate load path to transfer seismic forces exists, and the supporting columns can be demonstrated to have adequate capacity to resist the overturning forces generated by the shear capacity of the discontinuous elements.

Theoretically, it is desirable that the centre of mass and centre of stiffness of a building should coincide with each other, but in most cases, it is not possible to fulfil this criterion. During a seismic activity, the eccentricity between the two centres will induce a twisting moment and additional horizontal forces in a building. As a result of these twisting forces on a building, rotation of the diaphragm takes place, imparting additional seismic loads and lateral drifts onto some vertical members. The worst effect of torsion during an earthquake is on the columns that support the floor diaphragm. They are forced to drift laterally with the diaphragm inducing lateral forces and *P*-delta effects. Such columns often have not been designed to resist these movements and are extremely vulnerable to damage and collapse.

Buildings too close to each other may suffer from pounding (collision), which not only affects the dynamic response of both buildings but induces additional inertia loads on both structures. The pounding effect is much more serious if the floor levels of buildings do not match, i.e. they are at different levels.

Short columns are relatively stiffer than other columns in a storey and tend to attract higher seismic forces because of their high stiffness relative to other taller columns. If not adequately detailed, such columns suffer a non-ductile shear failure which may result in partial collapse of the structure. A short column with shear capacity to develop the flexural strength over the clear height will have some ductility to prevent sudden non-ductile failure of the vertical support system.

Approximate and quick checks are performed to compute the strength and stiffness of building components that primarily resist earthquake loads. The seismic base shear and storey shears for the building shall be computed in accordance with IS:1893 with the modifications as discussed above.

Shear stress in RC frames columns: The average shear stress in concrete columns τ_{col} computed in accordance with eq. (1) given below should be lesser of (a) 0.4 MPa and (b) $0.10\sqrt{f_{\mathrm{ck}}}$, where f_{ck} is characteristic cube strength of concrete:

$$\tau_{\mathrm{col}} = \left(\frac{n_{\mathrm{c}}}{n_{\mathrm{c}} - n_{\mathrm{f}}}\right)\left(\frac{V_j}{A_{\mathrm{c}}}\right), \tag{1}$$

where n_{c} is the total number of columns, n_{f} the total number of frames in the direction of loading, V_j the storey shear at level j and A_{c} the total cross-sectional area of columns.

Shear stress in shear walls: The in-plane shear strength of masonry walls is a crucial factor for the survival and stability of the unreinforces masonry (URM) buildings, particularly those buildings wherein the large size openings in masonry walls make them extremely weak and vulnerable. Average shear stress in concrete and masonry shear walls τ_{w} should be calculated according to the following equation:

$$\tau_w = \left(\frac{V_j}{A_{\mathrm{w}}}\right), \tag{2}$$

where A_{w} is total area of shear walls in the direction of the loading. For concrete shear walls, τ_{w} should be less than 0.4 MPa, whereas for unreinforced masonry load-bearing wall buildings, the average shear stress, τ_{w} should be less than 0.1 MPa. For reinforced and unreinforced masonry infills, the shear stress in walls should be less than 0.3 and 0.1 MPa respectively. This check on seismic robustness of infills is necessary for those cases in which the surrounding moment frames do not possess the required lateral strength. Consequently, the masonry infill can be considered as a primary load bearing structural element.

Axial stress check in moment frames: The maximum compressive axial stress in the columns of moment frames at the base due to overturning forces alone, F_{o} as calculated using the following equation, should be less than $0.25f_{\mathrm{ck}}$:

$$F_{\mathrm{o}} = \frac{2}{3}\left(\frac{V_{\mathrm{B}}}{n_{\mathrm{f}}}\right)\left(\frac{H}{L}\right), \tag{3}$$

where n_{f} is the total number of frames in the direction of loading, V_{B} the base shear, H the total height, and L the length of the building in the direction of loading.

Detailed evaluations are carried out for buildings with greater consequence in terms of loss of life. However, buildings with lesser consequences which have passed preliminary checks on seismic robustness are exempted from the detailed evaluation. A building must go for detailed evaluation if the following conditions are met:

(a) The building fails to comply with the requirements of the preliminary evaluation. (b) A building has six storeys and higher in RC and steel; and three storeys and higher in unreinforced masonry. (c) Buildings located on incompetent or liquefiable soils and/or located near (less than 12 km) active faults and/or with inadequate foundation details. (d) Buildings with inadequate connections between primary structural members, such as poorly designed and/or constructed joints of pre-cast elements.

The proposed methodology recommends detailed evaluation for all buildings with 'greater consequence in terms

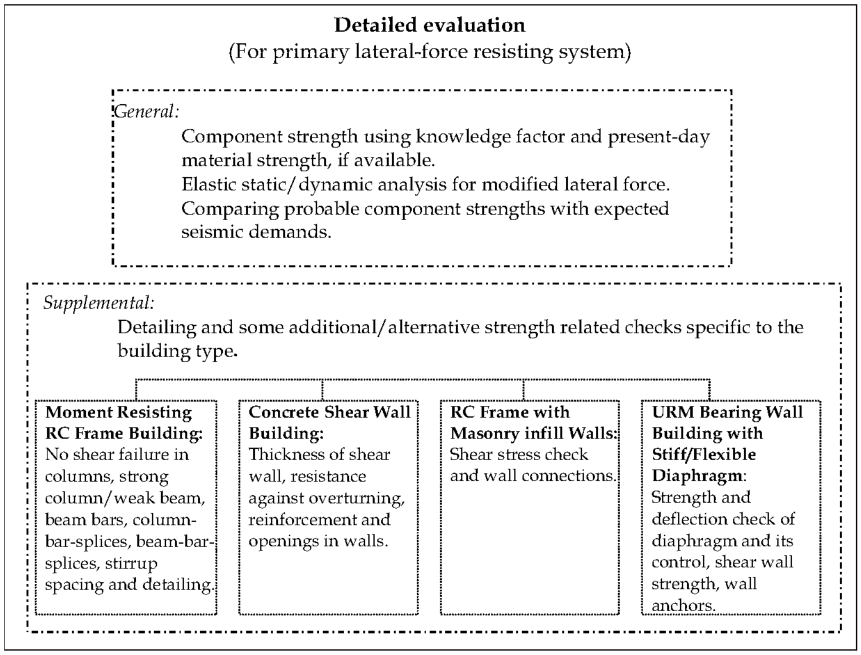

Figure 4. Various steps involved in detailed evaluation.

of loss of life'. A limit of six stroreys for RC and steel buildings, and three storeys for unreinforced masonry building is to keep the evaluation check simple for buildings with smaller consequences, which are adequately seismic robust as indicated by preliminary evaluation. Detailed evaluation will be essential when the decision to strengthen a building is taken at any stage.

The detailed evaluation procedure is based on determining the probable strength of all lateral-load resisting elements and comparing them with the expected seismic demands. An assessment of the present building condition and strength of materials is required. Further, seismic demand on critical individual components is determined using seismic analysis methods described in IS:1893 for lateral forces prescribed therein, with modification for reduced useable life condition. The procedure calls for a regular seismic analysis of the structural system of the building, followed by supplementary checks mostly related to detailing, suitable for a particular building type for ensuring satisfactory seismic performance. The steps involved are summarized in Figure 4.

During detailed on-site assessment, the building should be checked for the existence of some of the following common indicators of deficiency, such as deterioration of concrete, cracks in perimeter columns, deterioration in masonry units, and cracks in infill walls. In addition, evaluation of the present-day strength of materials should be performed using on-site, non-destructive testing and laboratory analysis of samples taken from the building. Standard test techniques used to assess present-day properties of structural material include usual tests, such as rebound hammer, ultrasonic pulse velocity, etc. However, for masonry, two tests have been developed relatively recently to evaluate compressive and shear strength *in situ* and are referred to as flat-jack test and shove test respectively. These techniques are especially useful for masonry buildings and are briefly described in Table 2.

The key steps of detailed evaluation procedure are described as follows:

(a) Estimation of probable flexural and shear strengths of critical sections of the members and joints should be performed according to the respective codes for various building types and taking into account the knowledge factor K.
(b) Design base shear is calculated in accordance with IS:1893 and multiplied with U, a factor for the reduced useable life. A linear equivalent static or dynamic analysis of the lateral load-resisting system of the building is performed in accordance with IS:1893 for the modified base shear determined in the previous step.
(c) Mathematical model of the physical structure is done so as to represent the spatial distribution of mass and stiffness of the structure to an extent that is adequate for the calculation of significant features of its distribution of lateral forces. Component stiffness to be used in the analysis is determined based on some rational procedure. Cracked flexural rigidity for non-prestressed beams and columns in compression can be taken in the range 25–45% and 70–90% respectively, of flexural rigidity computed on the basis of gross section properties.

Table 2. Standard test techniques for assessment of present-day strength of masonry

Test technique	Procedure
Flat-jack test – To determine engineering properties like *in situ* stress, compressive modulus, compressive strength and deformability of older and historic structures.	Two parallel gauge points are installed above and below the masonry wall. A horizontal saw cut is then made in a mortar joint between the gauge points. Gauge points move closer together as the masonry deforms and this distance is measured. A flat-jack is placed in the cut and pressurized, forcing the cut mortar to enlarge until the distance between the gauge points returns to initial values and the value of pressure is recorded, which gives a measure of the stress in the wall at the test area.
Shove test (in-place shear test) – To determine the average *in situ* mortar joint shear strength in existing unreinforced masonry.	It involves displacing a single masonry unit horizontally using a hydraulic jack. This necessitates the removal of a single masonry unit to provide access to the hydraulic jack and excavation of a head joint opposite the test brick to isolate it from the remaining masonry. The horizontal force required to cause the first movement of the test unit provides a measure of the mortar joint shear strength.

(d) Inter-storey drifts are calculated to decide whether it is acceptable in terms of the requirements of IS: 1893.

Acceptability of each component is evaluated by comparing its probable strength with the member actions under modified loads, i.e. using demand–capacity ratios.

A building is said to be acceptable if either of the following two conditions is satisfied along with supplemental criteria for a particular building type: (a) all critical elements of lateral force-resisting elements have strengths greater than computed actions and drift checks are satisfied, and (b) except for a few elements, all critical elements of the lateral force-resisting elements have strengths greater than computed actions and drift checks are satisfied. The design engineer has to ensure that the failure of these few elements will not lead to loss of stability or initiate progressive collapse. This needs to be verified by a nonlinear analysis, such as pushover analysis, carried out up to the collapse load.

In addition to the general evaluation for buildings which addresses only strength issues, further considerations are warranted which relate to ductility and detailing of structural components. These supplemental criteria address certain special features affecting the lateral load behaviour, which are specific to each building type. The provision of supplemental checks permits buildings with mixed structural systems to be evaluated; further, it keeps the procedure flexible, which can be expanded to include other structural systems.

For RC moment frame buildings if designed using response reduction factor R (IS:1893) equal to 5 (ductile detailing), some additional provisions of IS:13920 (ref. 13) need to be complied with. These provisions relate to shear capacity of frame members, anchoring of concrete columns into the foundation, moment of resistance of columns and beams, joint reinforcement, bar splicing and stirrups. On the other hand, if they are not designed for ductile behaviour, i.e. $R = 3$, they need not be checked against ductility provisions.

Concrete shear wall buildings can be either the ordinary reinforced type or ductile shear wall type. Shear wall thickness, resistance to overturning, longitudinal and transverse reinforcement and openings in the wall should be calculated according to IS:13920. RC frame with infill walls should be checked for the wall connections and out-of-plane stability in addition to provisions applicable to moment-resisting RC frame buildings.

Some additional provisions need to be satisfied for detailed evaluation of URM buildings with stiff diaphragms. These provisions relate to shear wall strengths in plane loading, out-of-plane forces for masonry walls, height-to-thickness ratio for masonry walls, wall anchorage and parapets. Provisions related to diaphragm connections, openings and reinforcement around diaphragm are also checked.

Unreinforced masonry buildings with flexible diaphragms should be checked for horizontal load transfer, strength and deflection[14]. Flexible diaphragm distributes the lateral forces to the vertical resisting elements on a tributary area basis, while a rigid diaphragm distributes lateral forces to vertical elements in proportion to their relative stiffness with respect to each other.

The actual seismic loading in the plane of a flexible diaphragm includes the distributed inertia force equal to the acceleration at the level of the diaphragm times its distributed mass. The ability of the floor and roof diaphragms to transfer lateral forces from the vertical resisting elements above the diaphragm to other vertical resisting elements below the diaphragm must be checked.

Deflection of a flexible diaphragm due to seismic inertia forces causes out-of-plane deflection of some walls. The out-of-plane walls are thus subjected to flexural stresses in addition to stresses due to vertical loads. Excessive deflection of a diaphragm can seriously undermine the load-carrying capacity of out-of-plane walls.

The magnitude of diaphragm deflection should be limited so that walls are not subjected to extreme and damaging deflections.

An appropriate level of seismic safety for existing buildings can be ensured only after careful evaluation of their ability to resist earthquake forces and strengthen them, if found deficient. The evaluation procedure follows a multi-layered approach with increasing level of complexity of analysis and investigation. At the preliminary level, the evaluation method looks for the presence and good condition of primary structural elements which are required to resist earthquake loads, alongwith some simple calculations to ensure their adequacy. A detailed evaluation includes more formal analysis coupled with assessment of present-day properties of materials for realistic estimates of member strengths. The method further includes supplementary checks for detailing issues unique to various structural systems which play a major role in seismic performance of the entire system, besides those related to strength only. The seismic evaluation methodology presented here is based on a prescriptive engineering approach similar to that in practice for new buildings; however, these may lead to more performance-oriented procedures in future, with the experience gained from its usage.

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Differential intracellular replication and virulence gene expression of *Listeria monocytogenes* isolates in human epithelial and endothelial cell lines

B. Dhanashree[1,*] and W. Goebel[2]
[1]Department of Microbiology, Kasturba Medical College, Mangalore 575 001, India
[2]Biocenter (Microbiology), Am Hubland, 97074 Würzburg, Germany

***Listeria monocytogenes* isolated from a case of spontaneous abortion in a pregnant woman (placental tissue), a diseased sheep (blood) and fish were compared with respect to their capability to invade and replicate in human epithelial (Caco-2) and endothelial cells (HBMEC). Intracellular expression of various PrfA-dependent virulence genes that are of relevance for intracellular replication was determined. Our data show significant differences in the invasion and replication of *L. monocytogenes* strains in epithelial and endothelial cells in contrast to the control strain, EGD. The observations correlate with the different efficiency of intracellular expression of PrfA-regulated virulence genes in the two cell lines, as analysed by real-time PCR.**

Keywords: Cell lines, intracellular replication, *Listeria monocytogenes*, virulence genes.

LISTERIA MONOCYTOGENES is a Gram-positive, facultative, intracellular, food-borne pathogen that has evolved highly sophisticated strategies to infect humans and disseminate from the intestines to the blood and further to the brain stem and the placenta causing septicemia, encephalomeningitis, brain abscesses and abortion, especially

*For correspondence. (e-mail: dbiranthabail@yahoo.co.in)