SEISMIC EVALUATION METHODS FOR EXISTING BUILDINGS

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ABSTRACT
Recent US Department of Energy natural phenomena hazards mitigation directives require the earthquake reassessment of existing hazardous facilities and general use structures. This applies also to structures located in accordance with the Uniform Building Code in Seismic Zone 0 where usually no consideration is given to seismic design, but where DOE specifies seismic hazard levels.

An economical approach for performing such a seismic evaluation, which relies heavily on the use of preexistent structural analysis results is outlined below. Specifically, three different methods are used to estimate the seismic capacity of a building, which is a unit of a building complex located on a site considered low risk to earthquakes.

For structures originally not seismically designed, which may not have or be able to prove sufficient capacity to meet new arbitrarily high seismic design requirement and which are located on low-seismicity sites, it may be very cost effective to perform detailed site-specific seismic hazard studies in order to establish the true seismic threat. This is particularly beneficial to sites with many buildings and facilities to be seismically evaluated.

INTRODUCTION
Recent US Department of Energy (DOE) natural phenomena hazards (NPH) mitigation directives (DOE, 1993) require the earthquake reassessment of existing hazardous facilities and general use structures. This applies also to structures located in accordance with the Uniform Building Code (UBC, 1988) in Seismic Zone 0 where usually no consideration is given to seismic design, but where DOE specifies seismic hazard levels (DOE, 1994).

Here an approach is presented for performing seismic evaluation of existing structures classified in Performance Category 2, which were not designed for seismic loading. The approach relies on the use of preexistent structure analysis results. Specifically, three different methods are used to estimate the seismic capacity of a building, which is a unit of a building complex located on a site considered low risk to earthquakes. For structures originally not seismically designed, which may not have or be able to prove sufficient capacity to meet new arbitrarily high seismic design requirement and which are located on low-seismicity sites, it may be very cost effective to perform detailed site-specific seismic hazard studies in order to establish the true seismic threat. This is particularly beneficial to sites with many buildings and facilities to be seismically evaluated.

The primary concern is the integrity of the building under the now newly mandated seismic loading. However, the complex may need be examined as a whole to correctly evaluate the demand and capacity of the building. The building is a reinforced concrete ordinary frame building and was built without any explicit consideration for seismic effects. Because it is in seismic zone 0, this is consistent with the underlying philosophy of the building codes at time of construction and the current UBC. Based on its use the building has been classified as NPH Performance Category 2 or PC2 (DOE, 1994).

Having been customarily designed against certain wind loads, the building has some inherent lateral-load resistance capacity. The current requirement on DOE facilities puts a new and different demand on the building to resist lateral load from earthquakes. However, because of the low likelihood of concurrent extreme wind and earthquake, DOE requirements, based on building codes, do not require load combinations...
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involving both the extreme wind and the seismic loads at the same time. The issue now is whether the wind capacity is sufficient for the new seismic load. This leads to the need to evaluate the strength capacity of the lateral load resistance and the seismic demand of the building.

EVALUATION APPROACH

The evaluation of the seismic capacity and demand follows the directive of DOE-STD-1020, Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities. In the following, the quoted numerical values should only be viewed as an example to demonstrate the procedures of approach only. They are not meant to quantify or qualify any specific building.

Because the building is of PC2, the general procedure is governed by the Uniform Building Code with the exceptions of the seismic zone factor or equivalently the peak ground acceleration and of the ductile design detailing. In the absence of a recent site-specific and DOE approved seismic hazard definition, it is necessary to use as the peak ground acceleration the bigger of the site-independent value specified by UBC and the DOE site-dependent value (See respectively Table C4 and C5a of DOE-STD-1020). However, for existing facilities it is permissible to perform the seismic evaluation using a natural phenomena hazard exceedance probability of twice the value specified for new designs. For the building, this makes the peak ground acceleration equal to 0.09g. Also because the site is in UBC Zone 0, the equivalent static method can be used to analyze the response regardless whether the building is regular or not.

(1) Seismic load demand

According to UBC, the total base shear, V, representing the demand of the lateral earthquake loading to the building is

\[ V = \frac{(Z I C W)}{R_w} \]

where \( Z \) is the zone factor, \( I \) is the importance factor, \( C \) is the amplification factor, and \( W \) is the total relevant weight of the building.

The values of \( I \) and \( Z \) are fixed by specifications in DOE-STD-1020-94 once the site and the PC of the building are identified. The weight of the building, \( W \), can be estimated in a straightforward manner. For the example building and site, the following values are deemed appropriate: \( Z = 0.09 \), \( I = 1.25 \), \( C = 2.75 \). The value of \( Z \) and \( C \) has been scaled to be consistent with the DOE specification.

The only item that requires a judgment call is \( R_w \). Factor \( R_w \) is a reduction factor to account for energy absorption capacity of the facility that results in element forces representing inelastic seismic demand. The use of \( R_w = 1 \) is always acceptable. For concrete ordinary moment resisting frames, \( R_w \) is 5. Such a value is valid for facilities built with good seismic detailing practice per UBC (1988 and later versions) along with reasonably uniform inelastic behavior; otherwise, lower values should be used. The directive DOE-STD-1020-94 requires for PC2 and below no special seismic detailing if \( Z \) is 0.11 or less. Since \( Z \) is 0.09 for the building, the detailing requirement for seismic design per UBC can be deemed formally met. Therefore, for this ordinary moment resistant frame building the use of \( R_w \) equal 5 is deemed justified.

Therefore the total horizontal base-shear force, \( V \), is 6.2% of the total weight under consideration.

(2) Seismic load capacity

The response, consequently the seismic load capacity, under the lateral seismic load is determined by the static force procedure of UBC. Three analysis procedures are employed and their results are compared. The objective of using three procedures of increasing complexity and decreasing conservatism is to utilize the graded approach and to provide a cross check on the reliability of each procedure.

Buildings are normally designed to resist horizontal loads, either due to wind or earthquake. The wind exerts loads on a building over surface areas. The building weight has no effect on the magnitude of the demand of the wind. The weight may even be beneficial in terms of resisting wind effect. This is different in case of earthquake loads. The seismic loads to a building are directly proportional to its weight. The heavier the building, the larger the seismic load it attracts.

Since both types of horizontal load are to be resisted by the same structural frame system, it is expected that any existing structural analysis report involving horizontal load will contribute to reducing the effort to evaluate the capacity of the building to the new seismic demand.

The usefulness of a preexistent seismic-load analysis report is self-apparent. The seismic capacity of the building is then already in that report in one form or another. And the capacity can be directly quoted after verification and modification to meet modern requirements, if necessary. The wind analysis report may not directly contain the seismic capacity of the building. Nonetheless, it contains the wind capacity, or indirectly the strength of the horizontal resistance of the structural frame. With this information and the weight distribution of the building, the seismic capacity can be deduced.

What may be lacking in a wind analysis report is the building weight distribution, which would be available in the dead-weight analysis report. If the weight analysis report is not available, the weight distribution of the building structure can be approximately reconstructed from the strength or moment capacity of the columns, which are used in the wind analysis to establish the capacity of the building.

In the following, the lesser of the reports, the wind analysis alone is assumed available, and its data are used in all the procedures, particularly in the 1D limit-load-type analysis.
One-dimensional limit-load-type analysis based on preexistent analytical data

The ultimate moment capacities of the reinforced concrete columns are directly available from 2D wind analyses. The axial weight load for each column can be inferred from the details of the corresponding moment calculations.

Models made of columns only are used to examine the various degrees of interaction among the building and the complex. In all, one end of the columns is fixed to the ground and the other to the imaginary rigid diaphragms at appropriate levels. From the moment capacities at the two ends of a column, its lateral-load resistance capacity is obtained using the equilibrium between lateral force and moments. The total lateral-load resistance capacity of, or the total horizontal force that can be taken by, any model is the sum of the resistance of all the columns contained therein. The capacity of each model is adjusted according to the assumed magnitude and direction of the interaction force. The corresponding bounding maximum acceleration that a model can resist is then calculated from the lateral force resistance capacity and the weight of the model.

Through the assumption of null interaction force, the analyses can show the capacity of the building isolated from the building complex. This capacity may be greater than the demand. But the building complex as a whole may have a substantially less capacity due to interaction. Also in the calculation of lateral-load capacity of a column, it is assumed that the moment capacities at both ends are fully developed. Thus the possible, but not necessarily always likely, maximum capacity for lateral load is derived.

Consider all the uncertainties involved in such simple models, it is not clear if the building can meet the code requirements under the specified seismic load, unless the bounding capacity of the complex exceeds the demand by a comfortable margin. To check this, a more elaborate independent elastic analysis of the building frame model may be needed and its procedure is presented below.

Two-dimensional elastic analysis by a general-purpose computer software

An elastic ANSYS (1989) model simulating the worst section of the building complex can be based on the preexistent analytical data and the engineering blueprints. The allowable for a working-stress type analysis is obtained by dividing the ultimate strength capacities by the factor of 1.7.

The seismic load to a building depends on the weight distribution of the building. The weight load at each end of a column is first inferred from the data, as before. This weight load is then transformed into a mass at an end. A horizontal force directly proportional both to the magnitude and to the height of the mass is then applied at each mass point. The magnitude of this force is finalized by a simple scaling such that the sum of these forces is the same as the total base shear.

V the building complex must resist. These forces can be hand calculated and subsequently fed to the ANSYS analysis. The columns and beams are modeled as STIF4 elements. The material is homogeneous and isotropic concrete capable of resisting tension also. The nominal dimensions are used without any consideration to the steel reinforcement. The effect of reinforcement is incorporated through the calculation of the capacities of the columns and the beams.

If the capacity exceeds the demand, then a reanalysis shall be done to include the effect of accidental torsion as required by the UBC. If the results still indicate comfortable margin, then a conclusion can be reached about the seismic adequacy of the building. However, to accurately include accidental torsion (and some other effects), it is better to use special-purpose computer program that is designed for this purpose.

If the capacity is below the demand even without accidental torsion. An analysis by a special-purpose program is still needed to provide a more complete documentation and to convince the building owner of the potential paper inadequacy of the building. The detailed analysis also provide the location and nature of the weaknesses of the building. Through these remedies may be proposed to the owner.

UBC-type three-dimensional analysis by a special-purpose computer software

A significant factor that may influence the capacity evaluation, but that is not explicitly included in the previous analyses, is the three-dimensional effect of the building frame. Thus the building may be reanalyzed using the modeling of the 3D space frame including accidental torsion. At the same time the capacity and demand of the building components are calculated per UBC in a more detailed and explicit manner. Because of the inclusion of accidental torsion, it is not known a priori what the consequence of modeling the 3D effects may be.

The computer software system ETABS (1992) can be used for the analysis. This software is selected because of its capabilities to do 3D analysis and more importantly to perform demand capacity checks following a variety of design and analysis codes, including the current UBC; i.e., ETABS has a post processing module (CONKER) that analyzes or designs the reinforced-concrete columns and beams. Other advantages of using ETABS include that the p-delta effect can be included in the global analysis of the building system and that the finiteness of the column-beam joint can be modeled.

Made-up of columns and beams, the ETABS model of the building consists of many levels. These beams and columns are treated as uncracked concrete that is homogeneous and isotropic and capable of taking both tension and compression. The effect of steel reinforcement is used only in establishing the interaction curves of the column capacity, but not in the
global structural analysis that defines the load to each column.

The capacity interaction curve between axial force and moment for each column is automatically generated by the module CONKER. The resultant demand capacity ratios for all columns at all levels are examined. These ratios provide the 3D distribution map of the capacity and demand of the complex. The lack of capacity and the potential strength, if any, are apparent in the map. Hence, the map provides clues to where retrofit may be needed, the feasibility of potential strength to be released by more sophisticated analysis methods, and whether interaction between the building and the complex is a concern.

There may be difference between the patterns of under capacity. In some places, the demand may exceed capacity at both ends of many columns. And in some other places, the demand may exceed capacity at only one end of a column while the other end has sufficient capacity (i.e., the sum of the demand capacity ratios at both ends is less than 2). Therefore, for the latter, there is certain undeveloped strength in the under-capacity columns. This lack of development of available strength may be a consequence of ETABS using a linear-elastic analysis. Unlike plastic analysis, a linear elastic analysis does not redistribute loads from a severely loaded portion to a less loaded portion. This, however, can only be fully confirmed by 3D plastic or limit-load analyses.

(3) Story-drift limitation (p-delta effect)

The seismic capacity is also controlled by the interstory drift limitation. Thus, the requirement for seismic evaluation consists of two parts: that the structure must have sufficient capacity, i.e., greater than the demand; and that the structure must not undergo large deformation. The latter requirement is to prevent loss of function and to maintain global stability. Both are considered satisfied, when the interstory or story drifts are less than code-specified limits. Alternatively, a p-delta analysis can be performed to examine the stability issue. The p-delta analysis option of the ETABS is used in the current study. This eliminates the need to apply the moment amplification for demand-capacity check and to deduce the story drift ratios as required by UBC.

The story drift is the displacement of one level relative to the level above or below. The story drift ratio is the story drift divided by the story height. The limitation on drift ratio depends on the height of the building and on $R_w$. They are the smaller of 0.03/$R_w$ and 0.004 for buildings.

Since $R_w$ is 5, the limitation on the story drift ratio is 0.4%. The calculated drift ratios from the 2D ANSYS and the 3D ETABS analyses shall be checked against this limit of 0.4%. If they exceed the limit the seismic capacity must be reevaluated.

(4) Seismic interaction

It is possible that falling objects from the complex may hit the roof of the building during an earthquake. The capacity of the building roof to resist such demand must be evaluated.

The typical roof panel can be treated as a beam under dynamic load and can be approximately modeled by an equivalent single degree of freedom (SDOF) system. The effect of the fallen objects is assumed uniformly distributed over the beam. For a uniformly loaded, fixed-end beam, the collapse mechanism is that formed by three plastic hinges. The effective resistance function of the SDOF system of the roof panel that relates the deformation $y$ to the resistance $R$ is bilinear, where the parameters, the initial slope, the maximum resistance $R_M$, and the initial yield deformation $y_{el}$, are described and given in Biggs (1964).

The underlying principle of the impact analysis is that the potential energy of the objects and the external work done by the weights of the fallen objects and the beam must be fully absorbed by the internal work done by the resistance of the SDOF system. The impact need not be ideally elastic, therefore not all the energy is transmitted to the roof. The energy loss upon impact depends on the ratio $\lambda$ of masses of the objects and the roof and is quantified as a reduction factor $K$ in Roskic's Formulas (Young, 1989). This factor is applied to the potential energy of the falling debris in the energy-work balance equation, reducing the effective falling height of the objects, as shown below:

$$ K(K_M m)g h + K_L (m + M)g y_{\text{deformed}} = R_M (y_{\text{deformed}} - y_{el} / 2) $$

where the mass and load factors $K_M$ and $K_L$ are given in Biggs, $g$ is gravitational acceleration, and

$$ K = \frac{1 + 13 \lambda}{35} \left( 1 + \frac{1}{2} \lambda \right)^2; \quad \lambda = \frac{m}{M} $$

For any given mass of the objects and the equivalent height they are falling from, the above equations yield the deformation of the SDOF system in terms of its initial yield deformation. This nondimensional displacement can then be compared to the ductility ratio allowed of the roof to ensure the integrity of the roof of the building.

CONCLUSIONS

An economical approach for the seismic qualification of existing hazardous facilities and general-use structures to recent DOE NPH mitigation directives has been presented. The key findings of the work are given below.

In the reassessment of structures it may be very cost effective to perform detailed site-specific seismic hazard studies in order to establish the true seismic threat. This is
particularly so in sites where many buildings and facilities are to be seismically evaluated. For example, at the Argonne site the seismic hazard specified in DOE-STD-1020-94 would be 12%. By performing a detailed site-specific seismic hazard study, a reduction to 4% was achieved, which is significant.

The structural interaction between the building and the complex may be important. The analysis of the building alone may not be conservative, and hence lead to unrealistic and unsafe conclusion. The whole building complex must be included in a sensible analysis. Three different analysis procedures were proposed for the seismic reassessment of existing structures. Included were a 1D limit-load analysis, a 2D linear-elastic analysis, and a 3D linear-elastic analysis. The use of three independent analyses is not mandatory, but they provide a graded approach and a reliable cross check on the validity of each analysis. Specific method (1D limit load) and commercially available computer programs (ANSYS and ETABS) are proposed for the analyses. But any reasonable and sound engineering analyses and computer programs in particular those utilizing the finite element technique are acceptable.

If all three procedures indicate the building has sufficient seismic capacity, then it can be so concluded. If some procedure indicates the building has sufficient capacity but some indicates otherwise, then the more detailed procedure shall have more say to the final conclusion. If all three procedures indicate that the demand exceeds the capacity in some portion of the structure, then the 3D analysis must be examined in detail.

The 3D analysis provides a much clearer picture of the seismic under-capacities, real or on paper. Thus if the analysis demonstrates that in the building the exceedance of capacity is small and localized for each of the affected columns, then the potential strength may be brought into action by load redistribution. However, this effect is not captured by the linear-elastic analysis and can only be fully demonstrated by a plastic analysis. Alternatively, the local under-capacity column can be upgraded using the information from 3D analysis to identify and guide the retrofit design.

It should be emphasized that local seismic under-capacities of structural members identified by analytical methods and models, which are limited in their scope, are not synonymous with the potential failing of the building in an actual earthquake. It should be treated a call for further resolution of the uncertainties and conservatism in the analyses both of the capacity and the demand. If the building indeed has sufficient capacity, but insufficient to cover the conservatism inherent in simple analyses, then more refined analyses, which will be more costly and time consuming, must be employed to confirm such capacity of the building. In a similar way, the over conservatism in defining the seismic load demand or even the maximum ground acceleration itself should also be reexamined.

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