METHOD OF PLASTIC HINGE JOINTS IN DESIGN PANEL BUILDING UNDER SEISMIC INFLUENCE

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ABSTRACT:

A method is developed to estimate the reliability of precast R/C panel buildings which will be subjected to actual recorded and artificially generated earthquakes. The panel building is represented by a spatial system of plane stressed elastic structural members - wall panels and panel slabs coupled in discrete nodes by nonlinear zero length hinged connections. For consideration of ductility ground and interaction of building with the soil there is developed a complex model of soil-interface-building system containing non-inertial soil in the form of nonuniform isotropic semi-space and building, connected to each other in the zone of contact by non-elastically deformable hinge nodes. Three-dimensional nonlinear dynamic analysis of the soil-interface-building system is implemented by the finite element method in the form of substructure using an implicit time integration scheme by the computer program BUILDING-NL. There is given an example of a five story precast R/C panel building with wide spacing 7.2-8.4 m. This study has taken into account the stochastic nature of the ground motion in Tbilisi region. The calculated values of reliability parameters at different levels of peak ground acceleration indicate that the response of the building is satisfactory. The developed method and obtained results can be used in seismic risk study for new buildings of examined type under design, as well as for existing panel buildings of old generation for future seismic activity.

KEYWORDS: Plastic, Joints, Building, Seismic, Influence, Reliability.

1. INTRODUCTION

The behavior of precast panel buildings depends on the performance of their constituent reinforced concrete constructions – vertical wall panels and horizontal panel slabs, connected to each other by vertical and horizontal key joints. The analysis of the earthquake consequences that occurred in various seismic zones all over the world has shown that projected for design seismic load precast panel buildings possess high percentage of survivance and successfully perform the primary objective of earthquake engineering – protection of the health and safety of occupants. It is to be noted that the characteristic damages from strong earthquake events are as follows: crack formation and opening, crushing of compressed concrete, yielding and rupture of some main reinforcement at connection regions and the presence of thin cracks in wall panels primarily on ground floors. Due to the sliding and rocking mechanisms the vertical and horizontal connections mainly shear and tension-compression deformations are affected and nonlinear behavior is highly concentrated at key joint locations and at the contact surface between soil and building.

Normally precast panel buildings with many stories and spacing are analyzed with simplified nonlinear models which neglect effect of spatial structural performance (Becker et al., 1980, Caccese and Harris 1987). Some of the existing analytical models (Rekvava, 1990, Astarlioglu et al., 2000) are suitable for modeling three-dimensional performance of panel buildings under seismic loading. The primary effort of these investigations focused on the inelastic behavior of structural joints and determination seismic response of structure with the rigid foundation without consideration of numerical reliability analysis.

In the last decade the concept of Performance-Based Seismic Design (PBSD) for structures subjected to seismic loading conditions was introduced. According to contemporary design codes and guidelines PBSD is an assessment procedure that is based on advanced, usually nonlinear dynamic analysis methods. In order to apply successfully such analysis procedures one or more scalar parameters are chosen as measures of the damage sustained. These parameters are also known as Engineering Demand Parameters (EDP). The most common EDPs are the maximum interstory drift ratios, the maximum plastic hinge shifts or the maximum floor accelerations, that will be used with fragility relations to determine performance of building systems and components. Fragility relations express in mathematical terms the likelihood that a component will sustain a specified level of damage when exposed to a specified level of EDP. Performance is defined as probable consequences of earthquake damage, including structural and nonstructural components, fatalities, injuries, and the costs of repairing, replacing and downtime.

This paper presents an analytical approach to predict the panel building actual behavior and the evaluation of building reliability under strong seismic excitation using some phase of PBSD.

2. DESCRIPTION OF THE PROPOSED METHOD

The practical approach to PBSD considers a ground motion Intensity Measure, structural response to calculate EDP, resulting damage analysis, which relates the EDPs to Damage Measures and calculation of Decision Variables, in terms that are useful to decision makers such as direct losses, downtime (or restoration time), and life safety risks (Moehle and Deierlein, 2004). In the following, based on these procedures performance-based operational reliability formulation is suggested, though performance measures that relate directly to business decisions have not been considered at this time.

2.1. Ground Motion Model

Recorded accelerograms may be used to represent seismic hazard at a site. In the probabilistic analysis of the dynamic response of the structure, a large set of ground motion records is needed. Herewith there is a scarcity of strong motion records for regions of Georgia. This concern gives rise to the use synthetic earthquake time histories to represent ground motion. For the present study, to describe seismic ground motion a set of discrete nonstationary Gaussian processes are used that differ from one another by dominant frequencies, duration and other parameters (Eisenberg 1976).

Each j element of this set or the ground acceleration time history $U_g(t,\omega_j)$ in the direction of x(1), y(2) and z(3) axes is simulated by the following expression

$$U_{gk}(t,\omega_j)=A_k(t,\omega_j)X_k(t,\omega_j)\sigma_k(t,\omega_j) \qquad (2.1)$$

where (k=x, y,z); ω_j = the dominant j-th process frequency, its boundary values ω_{min} and ω_{max} are assumed on the basis of empirical data; $A_k(t,\omega j)$ = $\varepsilon t e^{-\varepsilon_j t}$ is the normalized envelope deterministic function defined in terms of so-called Berlag impulse with fixed values ω_j; $\sigma_k(t,\omega_j)$ = root mean square value of acceleration; $X_k(t,\omega_j)$ = the stationary Gaussian time history process with zero mean that is characterized by function of correlation as

$$K(\tau) = e^{-\alpha_{jk}|\tau|}(\cos \omega_j \tau + \alpha_{jk}/ \omega_j \sin \omega_j|\tau|) \qquad (2.2)$$

The generation of the stationary acceleration time history is accomplished digitally using the algorithm of recurrent difference equations.

Thus, the stochastic model expressed by Eqn. 2.1 is completely determined with fixed values ω_j using three parameters: α = correlation coefficient, characterizing width of the spectrum; ε = determines the effective duration and process nonstationarity; σ = random process intensity that is defined by its dispersion.

The computation of the parameters of the maximum considered earthquakes (MCE) was performed using computer program "TBILISI'' at eight seism generating zones of Tbilisi region (100 km environment), that can reveal maximum seismic effect on the territory of the city.

For the determination of dominant periods of ground vibration T, duration of the intensive phase of vibration D, and PGA U_g^{max} of the expected seismic hazard for given site functional dependences was used (Rekvava 1994)

$$T,D,U_g^{max}=f(M,R) \tag{2.3}$$

Where M = earthquake magnitude; R = hypocentral distance.

As there is no available data concerning the width of earthquake spectrum for the surveyed territory, when plotting a model of seismic hazard α_j is taken equal to 0.5 ω_j, parameter ε_j is defined on the basis of the given duration of intensive vibration where $U(t)>=0.5U_g^{max}$ and is equal to $\varepsilon_j = 0.0159\omega_j$. Root mean square accelerations σ of synthetic accelerograms were obtained considering that $3\sigma = U_g$. Value of the dominant frequency is determined as $\omega j = 2\pi /T_j$. Thus calculated parameters for the generation of synthetic accelerograms, minimum values of hypocentral distance R and duration D of the intensive phase of vibration are summarized in Table 1.

Table 1. Parameters of regional synthetic accelerograms.

Number of zone	R, (km)	ω (sec $^{-1}$)	α (sec $^{-1}$)	ε (sec $^{-1}$)	σ (cm/sec ^{2})	D (sec)
1st group with M = 6						
12	10	34.88	17.44	0.56	91	2.51
17	11.2	34.88	17.44	0.56	87	2.66
16	14.1	33.05	16.52	0.53	77	2.99
7	26.9	27.3	13.65	0.44	51	4.12
2nd group with M = 6.5						
20	60.8	19.03	9.51	0.3	33	7.8
3rd group with M = 7						
11	25	19.62	9.81	0.31	120	6.29
4	42.7	17.44	8.72	0.28	74	8.23
30	86.3	14.6	7.3	0.23	32	11.7

2.2. Soil-Interface-Building System Modeling

The mechanical model of the soil-interface- building system containing the non-inertial ground in the form of elastic nonuniform isotropic semi-space and building, conjugated to each other in the zone of contact by nonelastic hinge nodes (Fig.1) is developed for consideration of the soil and building interaction. The precast panel building is represented by a spatial system of plane stressed elastic substructures-wall panels and panel slabs connected in discrete nodes by nonlinear ductile zero length hinged links. The ground is simulated by the ensemble of three-dimensional elastic finite elements and the selection of ground dimensions on the plan is realized proceeding from the condition that their change must not have influence on building stress-strain condition.

The conditions of interconnection as separation and sliding (constructive nonlinearities) on the interface between the building and surrounding soil are modeled by contact elements, not passing the tension strength to surfaces belonging to the building and soil. The contact element is assumed to have zero thickness and can be conceptually thought of as consisting of springs and Goodman joint element. Note that the Mohr-Coulomb yield criterion is used to simulate interface behavior.

Three-dimensional analysis of the soil-interface-building system with localized nonlinearities is performed by the finite element method in the form of substructure (Rekvava and Mdivani, 2007). The static condensation procedure is applied in order to define stiffness properties for substructures. Stiffness matrices of the polytypic substructures with rigid joints are determined once in dynamic analysis and they are constant during the seismic motion. Stiffness matrices of the conditional superelements drawn up out of substructures and set nonlinear connections are corrected considering changes of stiffness properties of nonlinear links, idealizing hinged key joints, on the basis of the employed in a design curve cyclic deformation of joints.

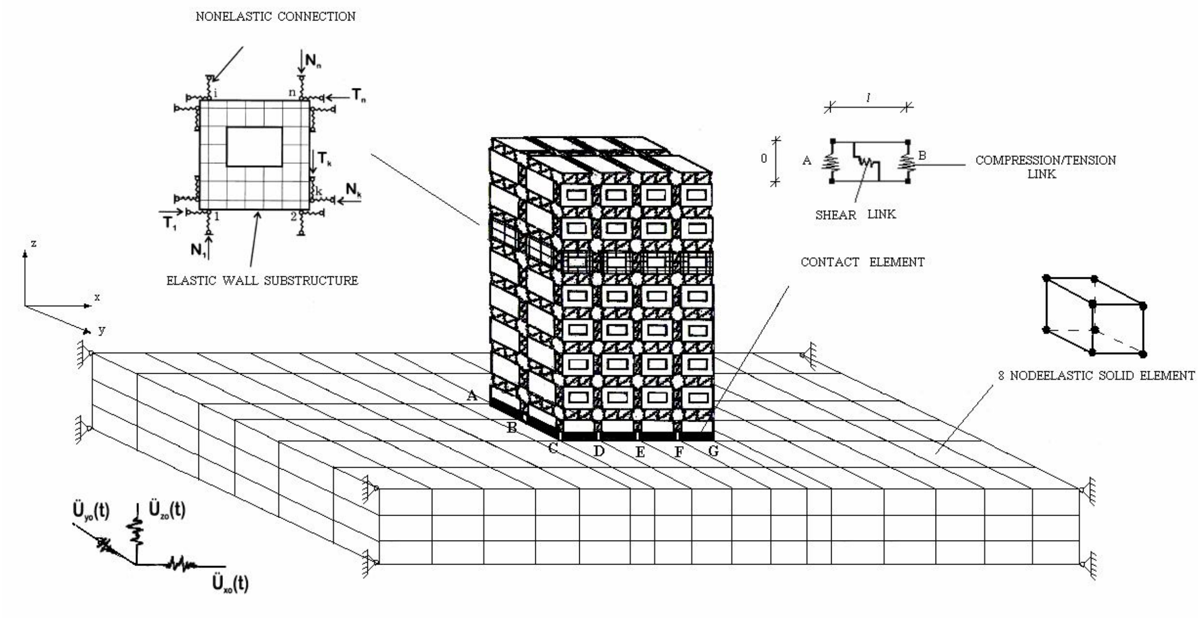

Figure 1.Diagram of the soil- interface-building model

The model of the building for dynamic analysis is idealized as a multi degree of freedom (dof) system consisting of masses. The acceptable dynamic results can be obtained by associating mass with only a limited number of dof and assuming that no inertia forces act in the other dof. Thus, each mass is lumped at the level of the floor at nodes of structures interaction and may possess three translation dof per node.

2.3. Hysteretic Component Modeling and Damage State Evaluation

For description of the strength and deformability properties of reinforced concrete vertical and horizontal key joints or of intersuperelement links in shear (T,a) and compression/tension (N,b) the polygonal curves including the effects of stiffness deterioration are shown in Figure 2.

The moment of entering into the phase of cracking, yielding and failure is specified by the von Mises yield criterion with the Mroz hardening rule that postulates a series of yield surfaces in a two-dimensional generalized force space. Each yield surface is represented by an ellipse and can be expressed as

$$f_l = T^2(t)/T_l^2 + N^2(t)/N_l^2 = 1 \tag{2.4}$$

where the subscript l=c,y,u refers to cracking, yielding and ultimate strength range; T(t),N(t) = current shear and normal forces due to both dead load and seismic load integral action at time t; T_l,N_l = ultimate shear and normal forces corresponding the above mentioned stages that are determined by the experimental results.

The reduction in the strength of key joints can be written as

$$Q_D(u_{mi}) = Q(u_{mi})\,(1-D) \tag{2.5}$$

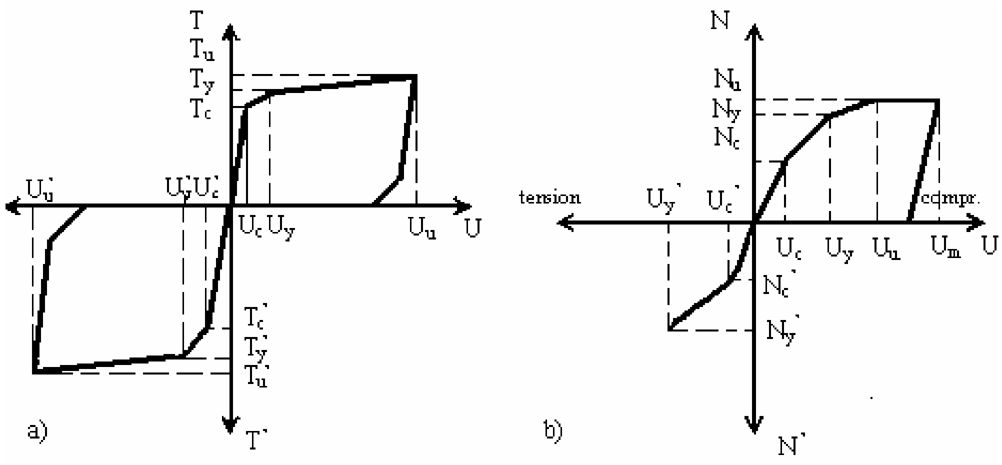

Figure 2. Hysteretic force-displacement relationships

Where u_{mi} = the maximum displacement the joint has experienced during the *i*th cycle; $Q(u_{mi})$ = the force corresponding to the displacement u_{mi} on the skeleton curve; D = the damage parameter whose value varies between 0 and 1, is a function of cyclic loading parameter γ and of velocity damage accumulation n (Wang and Shah, 1987).

Thus, prediction of damage in R/C joints is made in terms of cumulative fatique-type damage which is due to a significant number of inelastic cycles.

2.4. Nonlinear Seismic Analysis

Equations of dynamic motion for assumed model of the soil- interface-building subjected to earthquake ground motion at the time t can be written as follows (Rekvava and Mdivani, 2007)

$$MU(t)+CU(t)+F(t)=-MBU_g(t) \qquad (2.6)$$

where M = mass matrix of the model; C = damping matrix; U(t) = displacement vector; F(t) = stiffness (restoring) forces vector; B = matrix of coefficient of quasi-static effects of seismic influence; $U_g(t)$ = vector of the input ground acceleration time history, whose elements are given by the x-, y- and z-components of ground acceleration.

The incremental form of the equations of motion is obtained as follows:

$$M\Delta U_t+C\Delta U_t+K_T(u_t)\Delta U_t= -MBU_g(\tau)–(MU_t +CU_t+ F_t) \qquad (2.7)$$

where $K_T(u_t)$ = the tangent stiffness matrix of the model at time t, which is a function of the nodal displacements at time t; $\tau=t+\Delta t$.

The numerical integration of the nonlinear equations is performed employing the Newmark constant average acceleration method (β=1/4 and γ=1/2) with Newton-Raphson type iterative technique to achieve equilibrium at the end of each time step.

Note that to define the k^{th} iteration in the step from t to t+ΔU linearize the stiffness forces as

$$F(t+\Delta U)= F^k(t+\Delta U) + K^k_T(t+\Delta U))\Delta U \qquad (2.8)$$

The updated stiffness forces $F^{k+1}(t+\Delta U)$ are computed from $F^{k}(t+\Delta U)$ by following the actual nonlinear behavior through the increment. After convergence the next time step commences.

After sample response histories of sufficient size are generated statistics are taken on the significant response quantities to determine their probabilistic parameters, which are in turn used for the reliability analysis of the building by means of Monte Carlo techniques by the computer program BUILDING-NL.

For nonlinear dynamic analyses the initial conditions are represented by the deformed configuration under the influence of static (gravity) loads. Therefore, the first case is nonlinear static analysis of the building and the displacements, velocities, forces from the end of previous analysis are carried forward.

2.5. Reliability Formulation

The building failure criterion is considered the moment when the roof relative displacement value exceeds its reasonable one

$$| \delta/H | > [\delta/H] \tag{2.9}$$

where δ = the general roof displacement of the building; H = the building height; $[\delta/H]$ = allowed value of the given parameter which changes within the 1/1200 - 1/1500 ranges.

The reliability of structure R_s is evaluated on the basis of statistical method by (Rekvava, 1998)

$$R_s = 1 - P_o/P_t \tag{2.10}$$

where P_o = number of failure event which is connected with the fulfillment of the condition (2.9) under seismic influence; P_t = total number of roof displacement during seismic influence considered as a realization of random function.

Under the equal probability condition the following value of the reliability is defined

$$R_s = \frac{1}{n}\sum_{i=1}^{n} R_{si} \tag{2.11}$$

where n = number of seismic influence.

The seismic resistance criterion of the panel building generally is written

$$R_s > R_{ul} \tag{2.12}$$

where R_{ul} is the admissible reliability value and is adopted to be equal to 0.9-0.99.

3. ANALYSIS OF THE RESULTS

3.1. Building Description

The investigated panel building has 5 stories including wide (7. 2-8. 4 m) spacing and is assumed to be located in Tbilisi area under the medium ground conditions. The structure is regular in plan and vertically. The typical storey height is equal to 3.0 m. The dimensions of the building with spacing 7.2 m in plan are 27.0x18.0 m. The design loads adopted are: 1) the structure self-weight; 2) the design live load. The connections of panels are realized by reinforced key joints of heavy concrete (class B15, diameters of steel bars 2 * 12 mm for horizontal junction and 4*10 mm for vertical junction of class AI, modulus of elasticity E= 15500 MPa). The initial values of axial K_a and shear stiffness K_s for the vertical and horizontal key joints: K_a^V=432*10^4 kN/m, K_s^V=32*10^4 kN/m, K_a^H=313*10^4 kN/m, K_s^H=33*10^4 kN/m. The building is founded on the ground presented in appearance of a rectangular prism with sizes in plan 280 x 170 m. The ground segment from a surface to basic rock consists of two layers (H_1=10 m loam, E=58 MPa and H_2=50 m clay, E=33 MPa). The maximum values of stiffness for contact elements in shear and compression are k_z=36.5*10^4 kN/m and k_z=36.5*10^4 kN/m, respectively.

3.2. Seismic performance of the building

The calculation was carried out on the basis of design model considering the nonlinear ductility of connections of structural elements, the contact surface between the building and ground and the initial conditions (strained state from static load) at mixed system of bearing walls spacing in the first version – 4.2 and 7.2 m, and in the second version – 4.2 and 8.4 m. The recorded three-component accelerogram of the 2002 Tbilisi earthquake with scaled PGA of 0.2 g and three-component synthetic accelerograms generated using the data of table 1 for zones 12, 16, 7 and 11 were used. The peak horizontal accelerations re-counted to the rock of synthetic accelerograms corresponding to the zones compose 1.18; 1.09; 0.94 and 2.69 m/s^2, respectively. Performance objective associated with an earthquake with 2% in 50 year probability is accounted for in the design.

The initial fundamental period of the system vibration for both cases is equal to $T_1^{7.2} = 0.39$ sec and $T_1^{8.4} = 0.42$ sec, respectively. The analysis shows that increasing of the wall spacing up to 8.4 m conditions the increasing of initial basic tone of the period of natural vibrations per 5%, whereas the difference in the values of higher tones of period composes 30-90 %. During the elastic-plastic vibration of building with spacing of 7.2 and 8.4 m maximum horizontal relative displacement of roof and stories drifts are less then 1/1200 and 1/200, respectively that indicates the ability of a large-panel system with wide spacing under examination to resist to earthquakes of various spectral content. The value of slippage at elastic-plastic vibration on the interface of building-ground reaches its maximum at the spacing of 8.4 m at generated earthquake with the prevailing period 0.32 sec and composes 0.0006 m that is 2.8 times greater then the effect of the 2002 Tbilisi earthquake. The building maximum subsidence is 0.013 m that is less than the maximum allowable one for large-panel buildings. In consideration of real and generated accelerograms there is no disturbance of the contact along the vertical axis of the building.

The increase of spacing up to 8.4 m does not cause damage of the floor slabs with exhausting of carrying capacity of the compressed cross-section. The concentration of main tensile stress zones is observed in the joints of connection and exceeds the concrete design resistance in tension that conditions the local damages in these places. The maximum compression stresses in the panels of external and internal walls at increasing of spacing up to 8.4 m are raised per 19-30%, but they remain less then design compression resistance of the concrete of respective class. In the most strained panels of the first floor the cracks appear under action of accelerograms of zone 11, whereas the other earthquakes under consideration do not affect significantly the structure operation. The deformation of key joints of structural elements has a complex character. The number of elastic-plastic cycles of deformation depending on the duration and spectral content of real earthquake and generated accelerograms reaches 10-30. Cracks in the key joints and local damages are developed, but permanent displacements do not exceed the allowable ones in the horizontal and vertical directions. Here the normal (compression, tension) and shear forces are increased per 1.2-1.3 times in comparison with spacing of 7.2 m, and compose respectively 56% and 41 % of ultimate strength of indicated joints.

CONCLUSIONS

An enhanced mathematical model has been presented that is capable of accurate reproduction of the complete three-dimensional nonlinear behavior of the soil-interface-building system under strong ground motion in Tbilisi region and used to study the reliability of a residential 5-story building of new generation.

The bearing capacity of the panel building with the super-wide spacing is not exceeded. It resists the effect of an earthquake of high intensity and retains the ability of further deformation. The system ductility ratio of the building is equal to ν_B=2. The normative coefficient K_1 (1/q, where q is the behavior factor used for design in Eurocode 8) considering the structure capacity to develop the inelastic deformations, in this study composes 0.5, that indicates the low degree of nonlinear deformability of the building.

The building reliability for both versions are equal to 0.99 that is greater than ultimate allowable one for the dwelling houses that guarantees the structure safety from collapse and allows to recommend the expediency of experimental design and the construction of proposed building with super-wide wall spacing.

The developed method and obtained results can be used in seismic risk study for the new buildings of examined type under design, as well as for the existing panel buildings of old generation for future seismic activity and many decision-makers.

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