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Performance-Based Seismic Evaluation of Concrete Flat Slab Structures

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ABSTRACT

This paper focuses on the three existing single-story concrete flat slab structures of different vintages (1920's, 1950's and 1970's). These are roof structures of partially buried water reservoirs located in the Greater Vancouver area, and due to their function they are of post-disaster importance in case of a major earthquake. The main objectives of the paper are: i) to study differences in the seismic response estimate of otherwise similar concrete frame structures obtained using different analysis procedures; ii) to present a seismic performance comparison for otherwise similar concrete flat slab structures of different vintages, and iii) to evaluate provisions of the current Canadian Standard for Design of Concrete Structures CSA A23.3-1994 related to modeling of less than nominally ductile concrete flat slab structures, especially with respect to modified section properties and the force modification factor (R) values. Seismic evaluation of the three flat slab structures was carried out using the NBCC (1995) equivalent static analysis and the nonlinear static procedure ("pushover" analysis) as proposed by the FEMA 273 Guidelines for the Seismic Rehabilitation of Buildings. Seismic performance of the structures has been evaluated in terms of the lateral drift level, and the displacement ductility ratio; different reduction factors have been used to modify slab/column section properties in order to account for the effect of cracking. The results indicate that the NBCC equivalent static analysis offers a conservative estimate of lateral drift ratio and force modification factor as compared to the FEMA pushover analysis.

INTRODUCTION

Reinforced concrete flat slab is a simple structural system consisting of a slab supported directly by the columns. This system is favored by many designers due to the efficient use of materials and labor. It has been widely used in the construction of public buildings and partially buried concrete water reservoir roofs in North America. Consequently, there exists a large inventory of older flat slab structures that were designed to the codes which pre-date the present ductile detailing approach. Methodology for structural design of reinforced concrete flat slabs for gravity loads is well established, whereas under lateral loads uncertainty in the behavior of flat slabs persists. Due to a number of typical seismic deficiencies, it is generally feared that older flat slab structures will experience non-ductile, catastrophic failure when subjected to earthquake forces.

This paper focuses on three existing single-story concrete flat slab structures of different vintages (1920's, 1950's and 1970's). These are roof structures of partially buried water reservoirs located in the Greaterr Vancouver area. Due to their function these reservoirs are of post-disaster importance in case of a major earthquake. The main objectives of the paper are: i) to study the differences in seismic response estimate of otherwise similar concrete frame structures obtained using different analytical procedures; ii) to present a comparison of the seismic performance for otherwise similar concrete flat slab structures of different vintages, and iii) to evaluate provisions of the current Canadian Standard for Design of Concrete Structures CSA A-23.3-1994 as related to modeling of less than nominally ductile concrete flat slab structures, especially with respect to modified (cracked) section properties and recommended value of force modification factor (R).

LATERAL RESISTANCE OF CONCRETE FLAT SLAB STRUCTURES

Response of older flat slab structures to lateral loads was not studied until mid-1980s. Detailed experimental studies on the lateral response of concrete flat slab structures were carried out by Moehle and Pan (1988), Dovich and Wight (1994), and Durani, Du, and Luo (1995). The results of these research studies pointed to several critical parameters related to seismic response of flat slab structures, as discussed in the following text.

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Slab and column section properties. Current North American concrete codes (ACI 1995, CSA 1994) recommend that, "for lateral load, analysis of unbraced frames should account for the effects of cracking and reinforcement on stiffness of frame members" (CSA 1994). However, the Standard contains no explicit guidance for selecting these properties; instead, it requires reducing the effective stiffness of beam elements. The Standard recommends the equivalent section properties (moduli of inertia I) to be used in seismic design of concrete frame structures, i.e. $I_{column}=0.7xI_{gross}$, and $I_{beam}=0.4xI_g$. However, it also recommends a value of $I_{slab}=0.25x\ I_g$ for flat plate and slab structures (where I_g is based on the full slab panel width). ACI (1995) recommends: "for slabs of unbraced frames, an "equivalent width" in the range of 0.25-0.5 of the full panel width should be used to reflect reduced stiffness due to cracking".

Experimental studies mentioned above recommended a range of effective slab width coefficient values to be used for design purposes. Moehle and Pan (1988) concluded that the most appropriate value for the effective slab width coefficient for an uncracked slab section is in the order of 0.65 of the full panel width. This value should be modified by a stiffness reduction factor of 1/3 to account for the effect of cracking. Dovich and Wight (1994) recommend an effective slab width coefficient of ½ for strength analysis, and 1/3 for stiffness analysis (with respect to an uncracked section). In order to account for effects of cracking, slab and column stiffness should be modified by stiffness reduction factors of 1/3 and ¾, respectively.

Gravity load level. Results of several experimental studies have revealed that the level of gravity load on the flat slab is one of the most important parameters determining the lateral behavior of column-slab connections (Pan and Moehle 1998). Most failures in reinforced concrete flat-slab connections can be attributed to excessive vertical shear stresses induced by the gravity loads and unbalanced moments. According to Pan and Moehle (1988), the upper bound value of vertical (gravity) shear on a slab-column connection should be on the order of 40% of the punching shear strength, in order to ensure ductile seismic response of flat-slab structures. The study clearly showed that the ductility capacity of flat slab structures was significantly higher for the specimens subjected to low gravity shear loads. FEMA (1997) recognizes the importance of gravity load level in flat slab structures: it allows for higher ductility capacity for flat slab structures with gravity/shear strength ratio of less than 0.2, and prescribes non-ductile (elastic) seismic design for structures with gravity shear/punching shear strength ratio of over 0.4.

Displacement ductility ratio. Due to limited evidence related to lateral performance of flat slab structures, CSA (1994) states that "...the ability of such structures to sustain lateral loads when subjected to deformation reversals in the inelastic range has yet to be established. If used, they (two-way floor systems without beams) must be treated as a system with **R**= 1.5 and be subject to the restrictions of NBCC for such buildings." Therefore, the Canadian Concrete Standard recommends that flat slab structures should be treated as systems characterized with less than nominal ductility of 1.5 (nominal ductility systems are characterized with **R**=2).

FEMA (1997) recommends seismic acceptance criteria for two-way slab systems without beams designed using linear procedures. Depending on the level of gravity shear at the slab-column connections and the presence/lack of continuity of the bottom slab reinforcement, this document recommends value of *m* factor (which corresponds to R factor as per the CSA 1994) equal to 2 for structures subjected to low level of gravity shear load (gravity/punching shear ratio less than 0.2) for the Immediate Occupancy and Life Safety performance levels, and *m*= 3 for the Collapse Prevention performance level. The same document recommends m value of 1 (elastic response, corresponding to R=1) for the structures subjected to more significant gravity shear effects (with gravity shear/punching shear strength ratio value of over 0.4).

DESCRIPTION OF THE STRUCTURES

The three concrete flat slab structures studied in this paper are roof structures of partially buried water reservoirs located in the Greater Vancouver area. The structures are of different vintages i.e. Vancouver Heights (1920s), Kersland (1950s), and Central Park (1970s). Details of the reservoir structures are outlined by Sherstobitoff and Nikolic-Brzev (1999). It is believed that these structures were designed in compliance with the National Building Code of Canada editions current at the time of construction, and that seismic effects were not considered in the design (except for the 1970's structure). It should be noted that the three structures are subjected to low gravity load level, mainly consisting of dead load (self-weight); the roof area is mainly used as the ground for the public tennis courts, and hence only a nominal live load is prescribed for those structures. The review of the original construction drawings revealed a number of structural deficiencies related to lateral resistance of column and slab elements, which are considered to be typical for the older flat slab structures (see Table 1).

The three reservoir roof structures are regular in plan. Typically, the two-way roof slabs are thickened in the region over the columns with drop panels and tapered column capitals. Slab and column dimensions and amount of reinforcement in the three structures are summarized in Table 2.

Table 1. Seismic Deficiencies of Flat Slab Structures

	DEFICIENCY	FAILURE MODE
1	Discontinuous bottom slab reinforcement	Flexural slab failure at the slab-to-column joint
2	Lack of shear reinforcement at a critical shear perimeter	Punching shear failure in the roof slab
3	Inadequate lateral confinement of the column sections	Limited hinge rotation capacity
4	Inadequate development length of column reinforcement	Anchorage failure in the connections

Table 2. A Summary of the Structural Features

Vintage	Slab panel size (mm)	Column height (mm)	Slab thickness (mm)	Slab reinf. ratio[1] Top (%)	Bottom (%)	Column Size (mm)	Column Reinf. ratio (%)	f_c'/F_y[2] (MPa)
1920s	6,700	7,500	178	0.50	0.50	457(circular)	2.0	24/276
1950s	7,100	6,900	178	0.23	0.24	508(circular)	3.8	35/345
1970s	7,350	7,600	229	0.64	0.40	559 (square)	1.6	35/414

1 – at the slab-drop panel connection

2 – f_c'- concrete compressive strength, and F_y- steel yield strength.

The sites of all the facilities are located in the seismic zone IV of Canada according to the 1995 NBCC (1995). A set of performance-based seismic design criteria were custom-developed for these structures based on the three design earthquake events, as outlined by Nikolic-Brzev and Sherstobitoff (1999). However, it should be noted that, in the present study the performance of the three structures was evaluated for the Operating Basis Earthquake (OBE) event only. The OBE event was defined as a M7 to M7.5 earthquake with epicenter at a distance of approximately 30 to 70 km from the site, with an estimated firm ground PGA of approximately 0.19*g*, corresponding to a probability of annual exceedance of 0.0021 (a "475 year return period"); equivalent to NBCC 1995 design level earthquake. For the purpose of the Code-prescribed equivalent static analysis, the following parameters were considered: **v** = 0.2 (velocity factor), **F**= 1.0 (foundation factor, corresponding to firm soil conditions), and **I**= 1.0 (importance factor). Although the structures studied are of post-disaster importance, compelling the use of I= 1.5, in this study I value of 1.0 has been used; the latter value was considered more appropriate for the sake of comparison of the NBCC and FEMA seismic design procedures as applied to regular structures. According to the CSA (1994), value of **R**= 1.5 has been recommended for two-way slab systems without beams; R is force modification factor, which reflects the expected ductility capability of a structure. The NBCC-prescribed response spectrum curve has been used for the FEMA pushover analysis (in order to determine the target displacement value).

RESULTS OF THE SEISMIC EVALUATION

As all the three structures are regular in plan and elevation, typical 2-D frames have been identified and analyzed using the NBCC (1995) prescribed equivalent static (elastic) analysis and the nonlinear static procedure ("pushover" analysis) proposed by FEMA 273 (1997). Equivalent beam width model has been used in the analysis, as that model proved capable to simulate the lateral response of flat slab structures in the experimental studies (e.g. Moehle and Pan, 1988; Dovich and Wight, 1994). In the equivalent beam width model, slabs are replaced by beams spanning in the direction of lateral loading. Section properties for column and slab elements have been varied. A model with gross section properties includes gross column sections i.e. $I_{col}= I_g$, and half of the full panel width for the slab sections i.e. $L_{eff}= L/2$ (L_{eff} – effective slab width, and L - full slab panel width), whereas a model with cracked section properties includes $I_{col}=0.7xI_g$ for column sections, and Leff=L/4 for slab sections. Plastic hinge properties used in the pushover analysis were determined as per the FEMA (1997) recommendations for flat slab structures; flexural hinges were assigned to slab elements, whereas flexural-axial hinges were assigned to the column elements. Pinned-base column support condition were considered in the analysis, in order to account for an expected reduction in the level of rotational soil spring stiffness due to rather large earthquake-induced soil deformations; soil-structure interaction effects were not considered in the analysis.

The most relevant findings of the seismic evaluation are discussed below.

Cracked Vs Gross Section Properties. As indicated in the previous section, the three flat-slab structures have been analyzed considering cracked and gross (uncracked) section properties for column and slab elements. The results indicate that a variation of section properties (cracked vs. gross sections) influences the estimated seismic performance for the respective structural models. As an example, a chart illustrating drift ratio for the gross (uncracked) and cracked structural model of the 1920's flat slab structure is presented in Figure 1. Variations in the drift levels for the cracked and uncracked model are in the order of 25% (higher values are associated with the cracked section properties); this statement applies to the values of drift ratio determined using the NBCC linear static procedure and the FEMA pushover procedure-IO performance level. Variations of drift ratio are on the order of 10% for the FEMA pushover procedure- LS performance level. However, it should be noted that drift ratio determined based on the FEMA target displacement remains virtually the same irrespective of section properties used in the analysis, due to the fact that both the cracked and uncracked structure are rather flexible, characterized with the respective fundamental period values of 1.69 and 1.97 sec, respectively. A very similar trend has been observed for the other two structures, of 1950's and 1970's vintages.

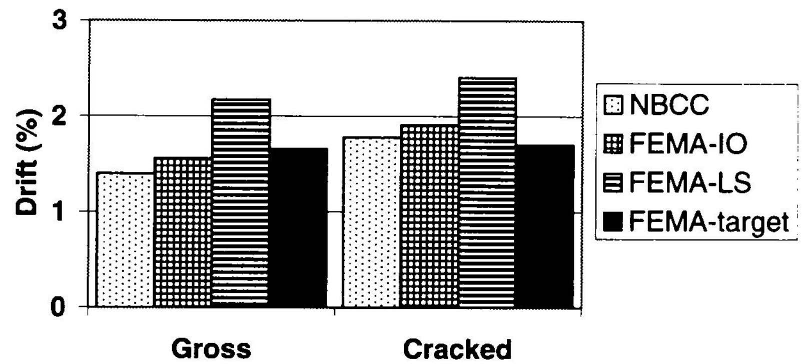

Figure 1. Lateral drift ratio for the 1920's structure – the effect of gross vs. cracked section properties.

Stiffness properties. The results of the modal dynamic analysis have shown that all of the evaluated structures are rather flexible. A chart showing the comparison of fundamental period values for the three structures (considering cracked section properties) is presented in Figure 2. It can be observed that the fundamental period values range from 1.27 sec (corresponding to the 1970's structure) to 1.97 sec (corresponding to the 1920's structure). It is also noteworthy that there is a trend of an increased stiffness (corresponding to a reduction in the fundamental period values) for the newer structures. A variation in the fundamental period values due to the variations in section properties (gross vs. cracked) has been observed. In general, a 15% increase in the fundamental period value in the "cracked" as compared with the gross-section structural model has been observed in all three structures.

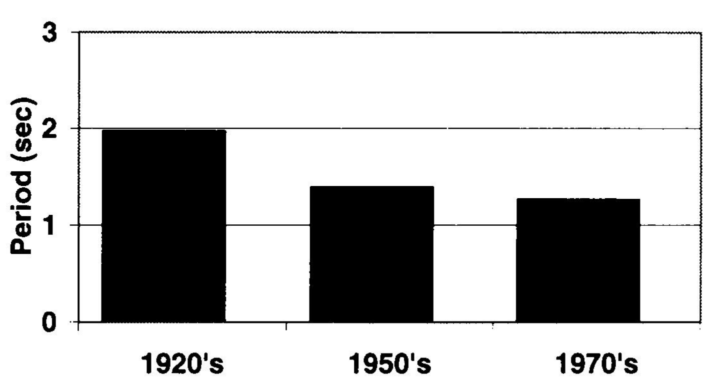

Figure 2. A comparison of the fundamental period values for the three structures (cracked section properties).

Lateral drift ratio. The evaluation has shown that, in general, seismic lateral displacements in all three structures remain within the limits prescribed by NBCC (1995). According to NBCC, a 1% limit for the drift ratio has been allowed for post-disaster structures, whereas a 2% limit has been prescribed for other structures. A chart showing lateral drift ratios for the three structures obtained using the NBCC-prescribed equivalent static procedure (NBCC) and the FEMA 273 (1997) prescribed pushover analysis is indicated in Figure 3a. It should be noted that FEMA-IO indicates lateral drift ratio corresponding to the Immediate Occupancy (IO) performance level obtained using the FEMA pushover analysis, whereas FEMA-LS indicates lateral drift ratio corresponding to the Life Safety (LS) performance. FEMA-target indicates lateral drift ratio corresponding to the target displacement determined as per the FEMA (1997) recommendations. The most relevant observations related to the drift levels obtained using these procedures are:

- Lateral drift levels obtained using the NBCC procedure are lower than the values obtained using the FEMA procedures in all cases; drift level determined based on the NBCC procedure thus represents the most conservative drift estimate, if compared with the estimates based on the pushover procedure.

- Drift levels that correspond to the FEMA Immediate Occupancy (FEMA-IO) performance level obtained in this study range from 1.14% for the 1950's structure to 1.91% for the 1920's structure. Except for the 1920's structure, the FEMA-IO drift levels are lower than the values corresponding to the target displacement determined as per FEMA (1997). It should be noted that the target displacement has been determined here assuming the Life Safety performance. In order to obtain the values of drift ratio corresponding to the target displacement at the Immediate Occupancy performance level, the FEMA-target values shown in the chart for need to be reduced by 10%. Drift levels corresponding to the FEMA Life Safety (FEMA-LS) performance level obtained in this study range from 1.47% for the 1970's structure to 2.4% for the 1920's structure. All FEMA-LS drift levels are higher than the values corresponding to the FEMA target displacement.

- Drift levels corresponding to the FEMA target displacement (FEMA-target) are intended to represent the maximum drift likely to be experienced during the design earthquake (FEMA 1997). Based on the findings of this study, drift level corresponding to the FEMA target displacement is lower than the value corresponding to the Life Safety performance (FEMA-LS) and larger than the value corresponding to the Immediate Occupancy level (FEMA-IO), as illustrated in Figure 3a. As FEMA target displacement represents the spectral displacement corresponding to the design earthquake, it can be concluded that, in a NBCC design earthquake corresponding to a Vancouver site, all three structures would perform in the range from Immediate Occupancy to Life Safety; this is in compliance with the NBCC implicit seismic performance objective - protection of life safety.

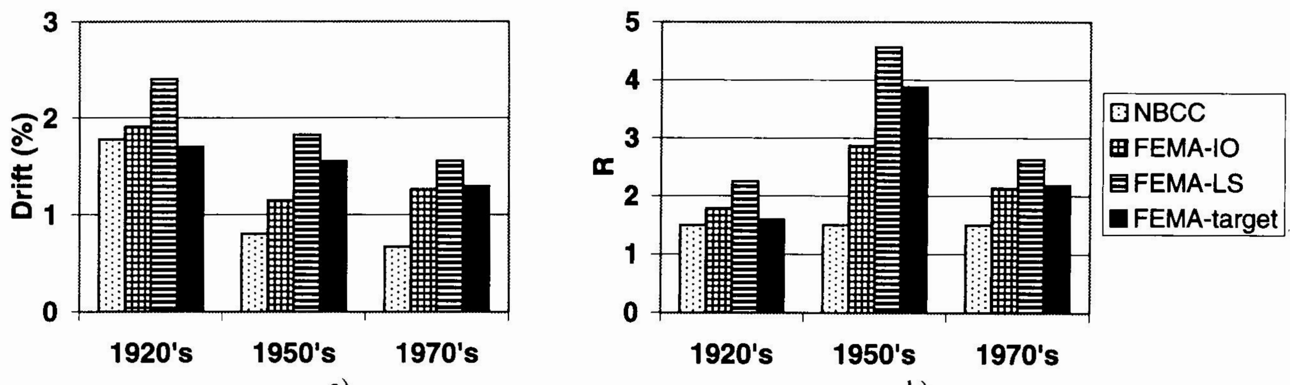

Figure 3. A comparison of the seismic performance parameters for the three structures (cracked section properties): a) lateral drift ratio, and b) force modification factor.

Displacement ductility ratio. The results also indicate that the estimate of lateral deformation capability, expressed in terms of ductility ratio (R), is different for the three structures considered in this paper. A chart showing the R values obtained using the NBCC equivalent static procedure and the FEMA pushover procedure is presented in Figure 3b. The most relevant observations related to the R values obtained using different procedures are summarized below:

- The force modification factor (R) values, prescribed by the CSA (1994) for concrete flat slab structures, are found to be conservative as compared with the values for the displacement ductility ratio obtained using the FEMA (1997) pushover analysis (R value of 1.5 has been used in the NBCC analysis procedure). Variations in the R values obtained using the NBCC analysis and FEMA pushover analysis range from 25 % (for the 1920's structure) to 150 %

(for the 1950's structure). Note that higher values are associated with the R ratios obtained using the FEMA procedure, and that an average of the three R values (for the FEMA-IO, LS and target levels) has been compared with the NBCC R value obtained using the NBCC static procedure. It should be also noted that R values related to the FEMA pushover analysis were determined as a ratio between the lateral displacement of a control node at a certain performance level (e.g. LS or IO) relative to the yield displacement of a same node.

- The R values corresponding to the FEMA Immediate Occupancy (FEMA-IO) performance level range from 1.79 for the 1920's structure to 2.86 for the 1950's structure. The R values corresponding to the FEMA Life Safety (FEMA-LS) performance level range from 2.25 (the 1920's structure) to 4.56 (the 1950's structure).

- The R values obtained using the FEMA pushover procedure, both at the Immediate Occupancy and Life Safety performance levels, are found to be greater than the FEMA (1997) recommended value for the force modification factor (*m* factor) of 2 for concrete flat-slab structures subjected to a low gravity load level and lacking a continuity of bottom slab reinforcement at the column locations.

Effect of the vintage. One of the objectives of this work is to develop a comparison of seismic response for otherwise similar structures of different vintages: the 1920's, the 1950's and the 1970's. The key observations related to the effect of vintage to the seismic performance of flat-slab structures are: 1) The older flat slab structures are found to be more flexible as compared with the newer ones (see Fig. 2); 2) Lateral drift levels in older structures are significantly higher as compared to the newer structures (see Fig. 3a), and 3) Displacement ductility ratio (expressed in terms of force modification factor R) corresponding to the older structures is found to be much smaller as compared to the newer structures (see Fig. 3b).

Gravity load level. The evaluation has shown that all the three structures are characterized with a low gravity load level; ratio of gravity shear to the punching shear strength is found to be less than 0.2; this has been taken into account while selecting the acceptance criteria for the IO and LS performance levels for slab elements according to FEMA (1997).

CONCLUSIONS

Seismic performance of the three older flat slab concrete structures of different vintages has been evaluated in the paper. Lateral drift ratio and force modification factor (R) have been used as the key parameters in this evaluation. The results of this study indicate that the NBCC equivalent static analysis offers more a conservative estimate of the lateral drift ratio and force modification factor R as compared to the FEMA pushover analysis. The performance of all three structures at a target displacement defined by FEMA could be rated as a Life Safety level; this corresponds well with the implicit seismic performance objectives of the National Building Code of Canada.

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