DESIGN OF A ONE-STORY PRECAST CONCRETE BUILDING FOR EARTHQUAKE LOADING

R.A. Spencer (I)
W.K.T. Tong (II)
Presenting Author: R.A. Spencer

SUMMARY

This paper presents a discussion of the seismic design of a one-storey precast concrete building. A quasi-static lateral force method of design is used, and a procedure to ensure that inelastic behaviour occurs in desired locations is explained. Both the design strength and the over-capacity of the connections must be considered. Connections that are expected to undergo inelastic deformations must have adequate ductility. Three different designs of precast shear walls are considered and the results of non-linear response analyses are summarized. Reasons for preferring one design are outlined.

INTRODUCTION

The design of precast concrete buildings is not covered explicitly in the National Building Code of Canada (NBCC), (Ref. 1) or in most other design codes. The design of a one-storey precast concrete building (Fig. 1) assembled from double-tee wall and roof panels (Fig. 2), using the NBCC provisions for reinforced concrete buildings, is described in detail in Ref. 2. The design of a similar building using a U.S. Code (Ref. 3) is described in Ref. 4. A somewhat different approach to the seismic design of this building, using NBCC procedures, is described here, and the results of non-linear response analyses of the building are discussed. The sesimic design is discussed for shaking in the N-S direction only.

DESIGN METHOD

Loads in precast structures must be transferred through adequate connections to the foundation. It is important to identify a complete load path for each load case, and to ensure that each connection is adequate for the combined load that it is expected to carry. Horizontal forces due to earthquakes are assumed to originate primarily in the diaphragm, due to its mass. For earthquakes acting in the north-south direction on the building discussed here, the north and south side walls, which are attached to the ends of the diaphragm members, (and which together with a frame along line B (Fig. 1) carry the diaphragm gravity loads to the foundation), also contribute to the earthquake forces in the diaphragm. The earthquake forces in the diaphragm are transferred by connections to the east and west end walls, which act as shear walls for this load case. The east and west

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(I) Associate Professor, University of British Columbia, Vancouver, Canada

(II) Graduate Student, University of British Columbia, Vancouver, Canada

walls must transfer shear from the diaphragm, plus a small additional shear force due to their own mass, to the foundation. This system of forces is shown in Fig. 3. Openings in the walls are ignored in the earthquake analysis.

Lateral Forces

The lateral forces to be used for earthquake design are found using the quasi-static analysis procedure given in Ref. 1. The total specified lateral seismic force at the base V, given by V=ASKIFW is found first.

For this building in zone 3, the acceleration ratio A is 0.08, and the product SIF is 1. W is taken as the weight of the roof plus the weight of the panels in the four walls above the mid-storey height, together with 25% of the design snow load, giving a total of 9415 kN. The value of K reflects the material and type of construction, and ranges from 0.7 for ductile frame buildings to 2.0 for unreinforced masonry. No value of K is specified for precast concrete, but the value of 1.3, which is given for "buildings of continuously reinforced concrete, structural steel or reinforced masonry shear walls" having limited ductility is often assumed to be appropriate for properly detailed precast structures.

In concrete design the specified lateral forces are multiplied by a load factor of 1.8 (Ref. 5) to give the factored lateral loads used in limit states design.

CONNECTIONS

The connections used to transfer shear forces between panels are shown in Fig. 4. They have #15M bars with an area of 200 mm^2 and a yield strength f_y of 300 MPa embedded in the flanges of the double-tees. Adjacent panels are joined using weld plates between connections.

The nominal shear strength of a conection under monotonic loading can be found as shown in Fig. 4, with $\phi = 1.0$ and a steel stress equal to the yield stress, f_y. Under reversed cyclic loading the connections will remain elastic and exhibit stable behaviour as long as the maximum load applied is less than about 80 to 90% of the monotonic strength. They can carry a few cycles of loading at a load equal to the monotonic strength, and then their strength begins to fall. Under reversed cyclic loading the strength of a connection in a panel of adequate thickness will stabilize at about 50% of the monotonic strength provided the displacements are not excessive. The strength of connections embedded in concrete less than about 150 mm thick will continue to fall under cyclic loading, and these connections will have very limited ductility.

When a limit states design procedure (Fig. 1) is used the strength of the building is found using the design strengths of its components. For precast connections the design strength is found by multiplying the nominal strength by a capacity reduction factor $\phi = 0.7$ (Ref. 2). The design strength of the connections in Fig. 4 is $V_d = 60$ kN.

Dowels

Dowel bars with f_y = 400 MPa are used to connect the wall panels to the foundation. One bar is grouted into each stem of each panel during erection. Resistance of the panels to sliding is assumed to be due to shear friction with all dowels contributing to the clamping force. The coefficient of friction is taken as μ = 0.4 and the capacity reduction factor is 0.7 (Ref. 2).

The dowels also contribute to the lateral strength of the walls by resisting overturning of the panels (Fig. 5). In this application the design strength of a dowel in tension is found using a steel stress of f_y and a capacity reduction factor ϕ = 0.7.

INELASTIC BEHAVIOUR

The use of the NBCC quasi-static analysis procedure (Ref. 1) with K = 1.3 implies that some inelastic behaviour is likely during a strong earthquake. For this building it is assumed that this can occur in the dowels and connections, while the panels themselves remain elastic. The building should be designed so as to ensure that the roof diaphragm remains elastic while any inelastic behaviour occurs in the connections or dowels in the walls.

It is assumed here that the capacity of a connection or dowel during the first cycle of loading into the inelastic range might exceed the nominal capacity, and reach a higher value, referred to here as the over-capacity. The over-capacity of a connection, found using a steel stress of 1.1 f_y with ϕ = 1.0, is V_o = 94 kN. The over-capacity T_o of a dowel in tension, which may be strained well into the strain hardening range, is found using a stress of 1.25 f_y and ϕ = 1.0. The over-capacity of a wall, expressed as a lateral force ΣF_o acting at roof level, is the sum of the over-capacities F_o of the individual wall panels. It is found by assuming that all the conections and the dowels in tension reach their individual over-capacities (Fig. 5). The weight of a wall panel, W_w, is 56 kN. F_w, that part of the specified lateral seismic force due to the mass of the panel above the mid-storey height, is 3.3 kN.

SEISMIC DESIGN

Seismic design begins with the design of the east and west walls. Each wall should have a design strength sufficient to carry a load of $1.8F_1$ (866 kN) acting at roof level, where F_1 (Fig. 3) is found by assuming that the part of the specified lateral seismic force V due to the mass of the roof and the upper part of the north and south walls acts at an eccentricity of 5%. The design strength of a wall is taken as the externally applied force acting at roof level, in addition to seismic forces of $1.8F_w$ per panel, when the dowels and connections in the wall develop their design strengths. Because inelastic behaviour is expected in the walls they must have adequate ductility when loaded beyond their design strength. Finally the over-capacity of each wall should be found, so that the roof to wall connections and the roof diaphragm can be designed to remain elastic when the ground acceleration is sufficient to cause inelastic behaviour in the east and west walls. Three different designs are considered for these walls. Their characteristics are summarized in Table 1.

Wall Design A

Design A has two connections per panel edge and one #10M dowel (bar area = 100 mm^2) per stem. This gives each wall a design strength of 1034 kN and an over-capacity ΣF_o of 1705 kN. The resistance to sliding of this wall, based on shear friction as discussed above with φ = 0.7 and steel stress f_y is only 358 kN. This wall would require some additional mechanism to prevent the wall from sliding on the foundation during an earthquake.

Wall Design B

Design B has one #25M dowel (bar area 500 mm^2) per stem. This gives a design strength of 831 kN, which is 4% below the required strength of 866 kN. This strength could easily be increased by using larger dowels at the ends. However to simplify the non-linear response analysis this is not done here. The over-capacity of this wall is ΣF_o = 1518 kN and its resistance to sliding due to shear friction is 1796 kN. This solution, with no embedded connections in the flanges of the wall panels, should have reasonable ductility and should be economical to construct.

Wall Design C

The design in Ref. 2 does not consider the weight of the panels in the east and west walls, and ignores the dowels when computing lateral design strength. This leads to 3 connections per edge. Dowels, designed to resist sliding by shear friction are #20M (area = 300 mm^2) except for the outside stems of the exterior panels, where #30M dowels (area = 700 mm^2) are used. Considering the dowels and connections the design strength of a wall is 1770 kN and the over-capacity is ΣF_o = 2931 kN.

Roof-to-Wall Connections

The connections between the roof diaphragm and the east or west walls should be able to transfer a force equal to the over-capacity of a wall without exceeding their design strength. This should ensure that these connections remain elastic during a major earthquake. In this building these connections must also be able to resist additional loads due to vertical bouncing of the roof panels. The use of 32 connections per wall allows one connection to be embedded in each stem of each wall panel, and gives a total design strength for lateral force of 32(60) = 1920 kN per wall. This exceeds the over-capacity of wall designs A (1705 kN) and B (1518 kN), and the connections should have sufficient strength to produce inelastic behaviour in a wall and a flexural failure in the outer flange of the roof panel if bouncing of the roof should occur. The design strength of the connections is less than the over-capacity of wall design C (2931 kN) and connections with a higher total design strength should be used to connect the roof to this wall.

Connections in Roof Diaphragm

The number of connections required between the roof panels is found by assuming that the diaphragm behaves like a deep beam and that no connection should be loaded beyond its design strength under the action of a distributed lateral load of intensity w (Fig. 6) which is large enough to load the connections between the roof and the east and west walls to their design strengths. Thus the minimum number of connections required on joint line 2 (Fig. 3) is (9/10) 32 = 28.8, with (8/10) 32 = 25.6 on line 3, and so on. The minimum number of connections on any joint line is taken as 15.

Chord Steel in North and South Walls

The chord forces that follow from the assumption of deep beam behaviour are carried as tension and compression forces at the roof level of the north and south walls. Use of the distributed lateral force w used above with $\phi = 0.7$ would required 3 #35M bars with $f_y = 400$ MPa and total area of 3000 mm^2, made continuous over the length of each wall using welded connections. These wall panels must also be connected to the ends of the roof panels with connections strong enough to transfer both the shears which result in the chord forces and the tension forces shown in Fig. 3. The roof panels must also be connected to prevent separation above the frame B (Fig. 1).

NON-LINEAR RESPONSE

The non-linear response of each design to the N-S component of the 1940 El Centro earthquake record was found using a method described elsewhere (Ref. 6). This earthquake, with a maximum acceleration of .32g, would be expected to produce inelastic behaviour in structures designed with K = 1.3. Non-linear behaviour of the connections and dowels is modelled as shown in Fig. 7, while the wall panels are assumed to be rigid. The foundation is assumed to deflect when the panels rotate due to lateral movement of the wall. Design C was analyzed with #20M dowels throughout, which reduces the calculated over-capacity of a wall to 2865 kN.

Results of this analysis are summarized in Table 1. The maximum displacements at roof level appear reasonable when compared to the limit of 0.005H, or 27 mm, suggested in the NBCC Commentary (Ref. 7) for lateral deflection under unfactored seismic loads. The relatively large displacement of design B arises because of non-recoverable extension in the dowels after they yield in tension, which gives pinched hysteresis loops for a wall as shown in Fig. 8. The maximum dowel extension is about 7 times the extension at yield.

The connections in the walls of design A would be subject to a number of cycles of inelastic behaviour with a maximum displacement of about twice their yield displacement. Because the thickness of the flanges in which they are embedded is only 50 mm the ductility will probably be inadequate. Design A is therefore a poor solution to the seismic design problem. The connections in the walls of design C would barely yield, and the behaviour of these walls would be acceptable.

The maximum forces acting at roof level agree well with the calculated over-capacities of the walls. For design C this force exceeds the design strength of wall to roof connections, as expected. This design should be rejected because of possible inelastic behaviour in the roof diaphragm and wall to roof connections.

CONCLUSION

There are two important considerations which may easily be overlooked in the seismic design of precast concrete structures. Strengths of connections should be adjusted to ensure that inelastic behaviour takes place in the desired locations, and the ability of connections to withstand the expected reversed cyclic loads should be assured. Design B meets both these criteria, and in spite of larger predicted deflections under earthquake loading is the best solution to the seismic design problem for the building considered.

References

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2. "Metric Design Manual", Canadian Prestressed Concrete Institute, Ottawa, 1st Edition, 1982.
3. "Uniform Building Code", International Conference of Building Officials, California, 1976.
4. "PCI Design Handbook", Prestressed Concrete Institute, Chicago, 2nd Edition, 1978.
5. "Code for the Design of Concrete Structures for Buildings, CAN3-A23.3-M77". Canadian Standards Association, 1977.
6. W.K.T. Tong, "Non-Linear Seismic Analysis of a One-Storey Precast Concrete Building", M.A.Sc. Thesis, University of British Columbia, October 1983.
7. "The Supplement to the National Building Code of Canada 1980", Associate Committee on the National Building Code, National Research Council of Canada, Ottawa.

Table 1. Non-Linear Response Results for N-S Component of El Centro 1940 Earthquake

	Design		
	A	B	C
Connections per wall panel edge	2	0	3
Dowel bars per wall panel	2 - #10M	2 - #25M	2 - #20M
Max. displacement at roof level	15.7 mm	36.5 mm	9.8 mm
Max. force at roof level per wall	1703 kN	1507 kN	2794 kN
Calculated over-capacity ΣF_0 per wall	1705 kN	1518 kN	2931 kN

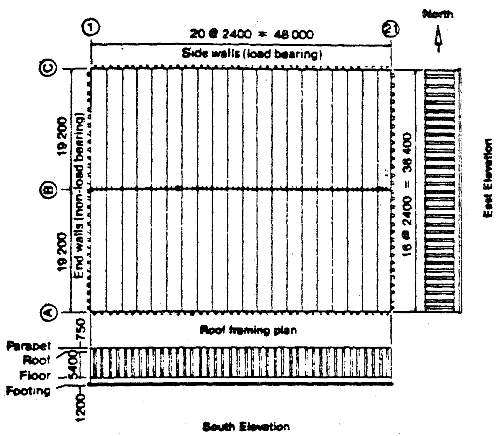


Figure 1: One-storey precast concrete building.

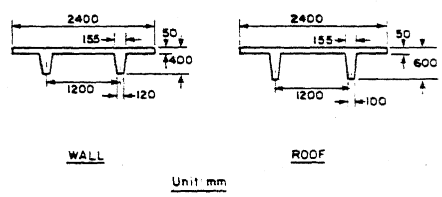


Figure 2: Dimensions of Wall and Roof Double Tees.

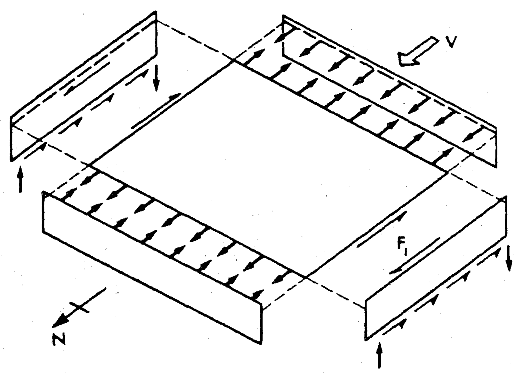


Figure 3: Seismic Forces in connections between Roof, Walls and Foundations.

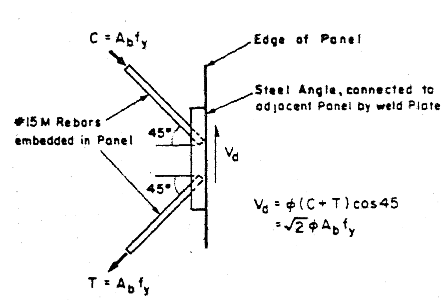
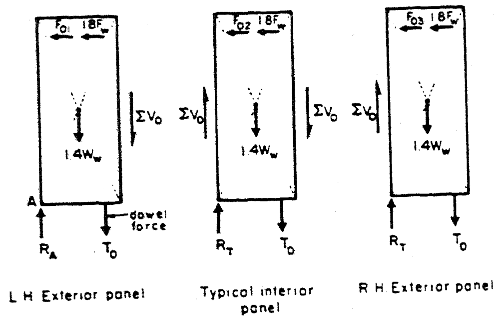


Figure 4: Embedded connection used to transfer Shear Force between Panels.



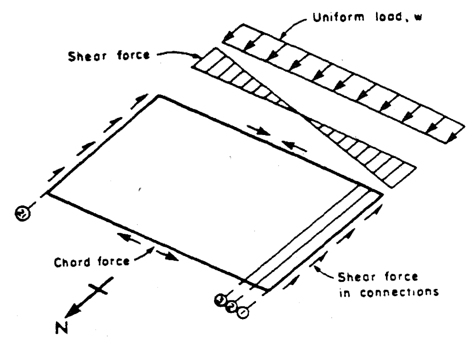


Figure 5: Resistance of East & West Wall panels to lateral loads. Figure 6: Forces assumed to act on Roof diaphragm.

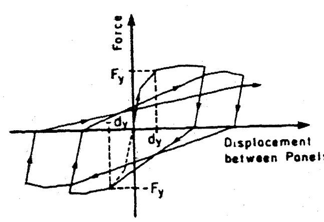


Figure 7(a): Hysteretic Model for Vertical Connection.

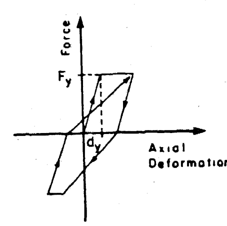


Figure 7(b): Hysteretic Model for Dowel Connection.

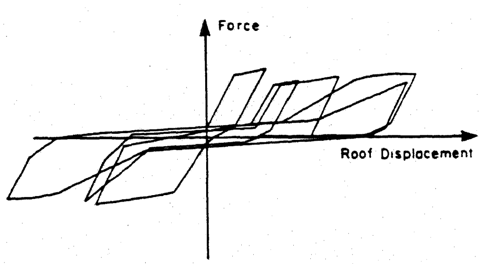


Figure 8: Hysteresis Loops for Building design B.