is to say, the added thickness of a drop panel, extending a distance at least L/6 into the span in question, is allowed to provide an increased moment arm within the drop panel region. For a rigorous computation, the rational approach shown in part (c) of Fig. 2.4.4-2 may be used.

Also note that, in the ACI Code, a slab thickening can qualify as a drop panel on one side of the column, but as a drop cap on the other side. In other words, the drop panel designation does not necessarily cover the entire thickening. Rather, it describes the regions of thickening on each side of the support separately.

Another simplified analysis and design procedure for the treatment of drop caps, commonly used by many engineers prior to the widespread use of computers is illustrated in Fig. 2.4.4-3. In this simplified procedure, the portion of the drop cap falling outside the 45 degree inverted, truncated cone, extending from the bottom of the cap at the column face to the extended soffit of slab (part (a) of the figure), is disregarded. Hence the geometry modeled in design is reduced to that shown with thick lines in part (b) of the figure. The truncated cone is considered part of the column, as opposed to the slab; the slab is analyzed with uniform thickness "h" throughout. The moments calculated at the system line (column centerline) are then reduced to the face of the truncated cap (W in part (b) of the figure) and resisted by a slab of thickness "h". The punching shear is checked at a distance equal to one half of the effective depth of the clear slab from the face of the truncated cap (not the original drop cap) and is resisted by an effective depth of the clear slab. In other words, the region falling outside the truncated cone, shown with broken lines in part (b) of the figure, is not utilized for either the moment resistance or the punching shear capacity. No punching shear check is performed within the cap when this model is used.

2.4.5 SUPPORT CONDITIONS

For gravity loading, it is commonly assumed that the response of a floor system can be adequately analyzed by considering the floor as an isolated level bounded by a single story of upper and lower columns/walls. Unless detailed otherwise, the columns are assumed fixed at their connection to the floor and at their far ends, away from the floor being considered.

Under certain conditions, the assumption of full fixity at the column-slab junction results in an impractical design, because the column reinforcement necessary to resist the computed moments become excessive, or the share of moment to be resisted through punching shear leads to unreasonable computed stresses. In such cases, engineers generally assume a reduced stiffness for the column in the design. This assumption is complemented by detailing the column connection to the slab with additional, closely spaced ties to minimize the width of cracks which may develop at the column/floor or column/beam junction. The ties are placed over a length equal to one and-one-half times the larger cross-sectional dimension of the column. Typically, 10mm bars at 80 mm spacing or less are adequate (#3 @ 3 in. Spacing). The two most common cases where a reduced column stiffness is assumed are outlined below:

A. Connection at uppermost level of long-span beams

Fig. 2.4.5-1 illustrates the framing of a parking structure. For 20m (65 ft) spans, the beams are typically 750 mm (30 in.) deep. The columns are typically square 500mm (20 in.) and the clear height of column is 2150mm (7 ft). In this structure the relative stiffness of the beam to the column is approximately 2 to 1. If the design of the uppermost column/slab connection is based on the gross moment of inertia of the beam and column, the computed moment at the joint will be too large to be resisted by the column. The situation is further aggravated by the fact that, at the roof level, the contribution of axial loading to column performance is small. Therefore the columns are assumed hinged at the locations shown in Fig. 2.4.5-1(b). Added ties described above provide the confinement required to allow rotation without undue cracking.

At lower levels, the moment at the beam/column connection is shared by the columns above and below. Further, increased axial loading in the column improves the performance. Hence a hinge detail is generally not necessary at these locations.

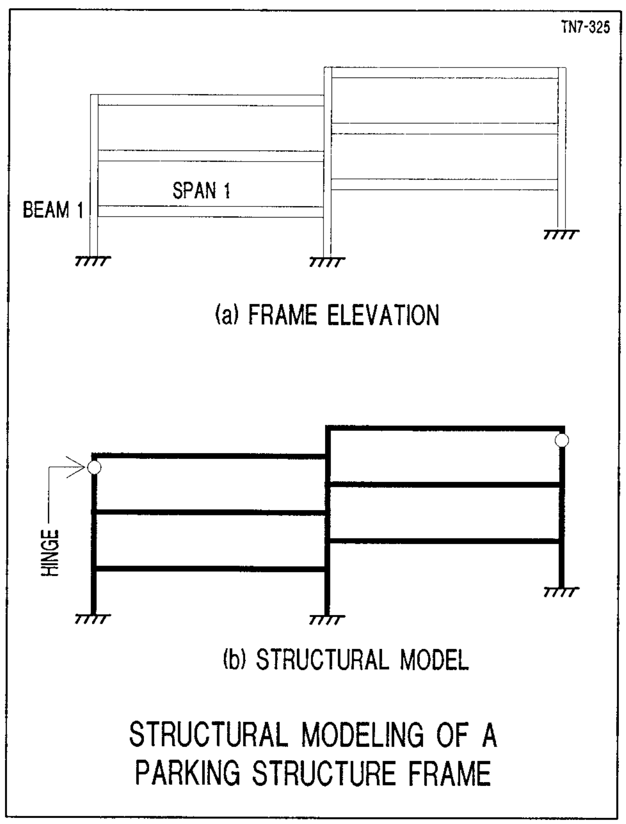

FIGURE 2.4.5-1