Design Live Loads for Passenger Cars Parking Garages

By Y. K. Wen,[1] Member, ASCE, and G. L. Yeo[2]

Abstract: The objective of this study is to obtain a better understanding of live loads in passenger cars parking garages and to recommend an appropriate value for design. Load surveys were conducted in nine parking garages in Chicago and Urbana-Champaign, Ill., and Boston and Cambridge, Mass. Based on the survey data, statistical analyses were carried out on the equivalent uniformly distributed loads, which produce the maximum column axial loads and beam midspan moments that the building is expected to experience during its lifetime. The dynamic amplifications due to vehicle motions on the decks and ramps were also investigated. The results suggest a possible reduction of the design live load from the current value of 2.39 kN/m^2 (50 psf), as stipulated in the ASCE-7-95 and other codes and standards, to 1.92 kN/m^2 (40 psf), but with no allowance for reduction according to the bay area.

INTRODUCTION

As stipulated in ASCE-7-95 (ASCE 1996), the *Standard Building Code* (Southern 1997), and the *Uniform Building Code* (International 1997), the design live load intensity that applies to passenger cars parking garages is 2.39 kN/m^2 (50 psf). No reduction according to area is allowed in ASCE-7-95. Provisions of both the *Standard Building Code* and the *Uniform Building Code*, however, permit a reduction of live load intensities in passenger vehicle garages. For example, the *Uniform Building Code* allows the design live load to be reduced at a rate of 0.08%/sq ft of the area supported for structural members supporting 13.93 m^2 (150 ft^2) or more. The *Uniform Building Code* allows the reduction of design load up to a maximum of 40% in horizontal members. The *Uniform Building Code* also stipulates that the reduction percentage shall be less than the value computed by the following equation:

$$\text{reduction percentage} = 23.1(1 + \text{dead load/live load}) \quad (1)$$

For most parking garages, the 40% limit implies a design load of 1.44 $\times$ N/m^2 (30 psf). There are several concerns regarding the above provisions. First, unlike live loads in office and residential buildings, which are generally spatially random, parking garage loads are due to vehicles parked in regular patterns and the garages are often full. The rationale behind the reduction according to area for other live loads therefore does not apply. Second, it is generally felt that the design live load of 2.39 kN/m^2 (50 psf) is overly conservative and that a lower value can be used without sacrificing structural integrity. This observation can be demonstrated by a simple calculation using the most conservative assumption of vehicle weight. To arrive at a more realistic design load and possibly justify a reduction of the current design value, a study of the parking garage vehicle loads and load effects on the structural members is necessary. A literature survey indicated that very little has been done in this area. For this purpose, a live load survey was carried out to collect data from nine major parking garages in downtown Chicago and Urbana-Champaign, Ill., and downtown Boston and Cambridge, Mass. Distributions of vehicle weights were established, and statistical analyses were carried out on the column axial loads and beam midspan moments that the structure is expected to experience over its lifetime. The corresponding equivalent uniformly distributed load (EUDL) that will result in the lifetime maximum load effect can then be calculated for possible use in code provisions. Details can be found in Wen and Yeo (1999).

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LOAD SURVEY AND ANALYSIS OF LOAD EFFECTS

Conduction of Survey

A load survey was conducted in parking garages in Chicago and Urban-Champaign, Ill., and Boston and Cambridge, Mass. To ensure an accurate representation of the cross section of vehicle load distributions in different buildings, multistory parking garages in buildings serving various purposes were surveyed. They are as follows:

1. Huron Plaza, Chicago (residential complex)
2. John Hancock Center, Chicago
3. Ontario Place Commercial Parking Garage, Chicago
4. Holiday Inn, Chicago
5. University of Illinois at Urbana-Champaign Rental Parking Garage, Champaign, Ill.
6. University Inn, Champaign, Ill.
7. Carle Medical Center South Clinic, Urbana, Ill.
8. Massachusetts Institute of Technology Parking Garage, Cambridge, Mass.
9. PI Valley Commercial Parking Garage, Boston

Each survey was conducted once. The extent of occupancy in the parking garage at the time of the survey was recorded. The sizes of each parking garage and each individual parking lot were measured. The make and model of each vehicle were also recorded. The weights of individual vehicles were then obtained from the *Market Data Book* [different editions of Automotive News (1998)], which contains specifications for cars available in the United States. In all, a total of 364 vehicles were surveyed in the state of Illinois, and 155 in Massachusetts; their curb weights ranged from 769.5 kg (1,693 lb) (Geo Metro) to 3,909 kg (8,600 lb) (Chevy Suburban). Passenger and cargo weights will be added in the analysis.

Distribution of Vehicle Weights

Statistics and histograms of the vehicle weights in each of the nine garages surveyed in Illinois and Massachusetts were checked to see if there was any dependence on geographical location and city size. Based on the mean and standard deviation of the vehicle weights, and on the shape of the histogram obtained for the vehicle weight distribution, we conclude that there is no significant difference among the car types that are present in the four cities of different sizes and locations. The distribution of the weights of 519 vehicles for all garages combined is shown in Fig. 1. The asymmetrical nature and positive

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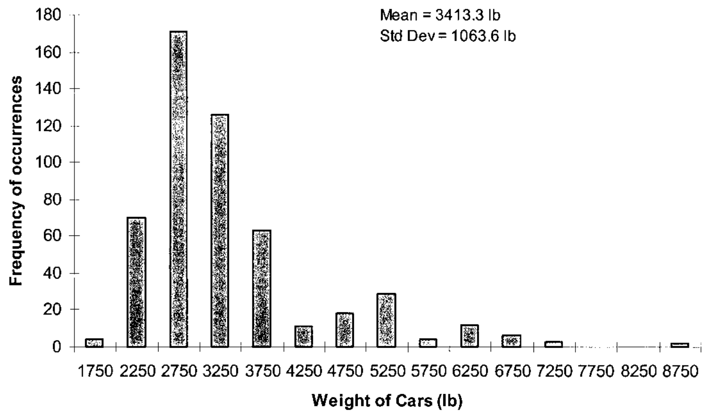

FIG. 1. Histogram of Parking Garage Vehicle Weights (1 lb = 0.454 kg)

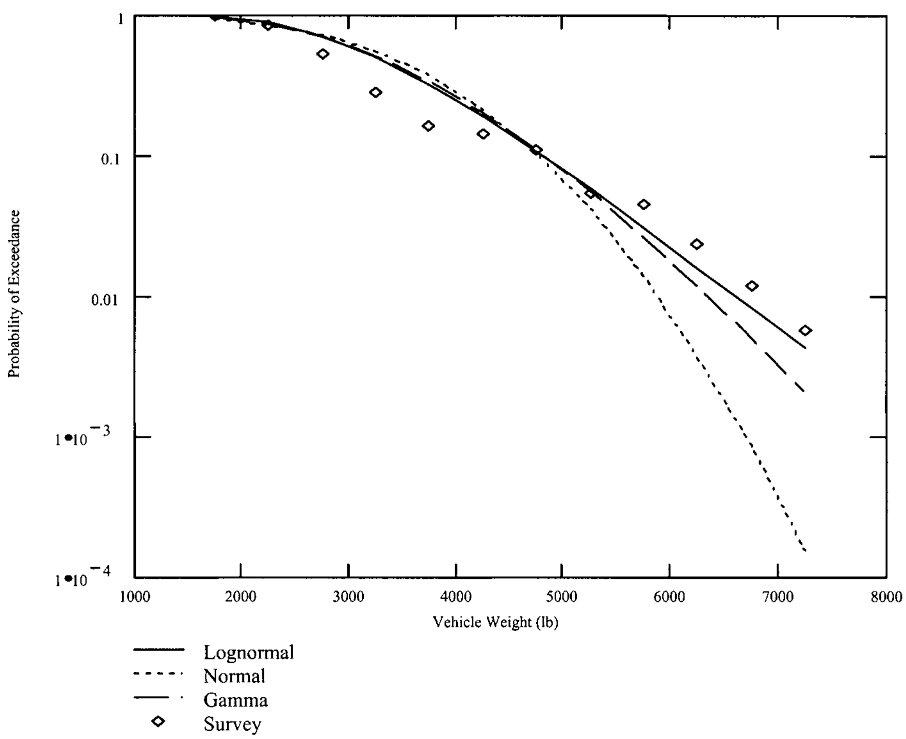

FIG. 2. Probability Distribution of Parking Garage Vehicle Weights (116 = 0.454 kg)

skewness of the histogram suggest that the lognormal distribution is a possible model. The values of the lognormal distribution parameter ζ and λ can be calculated from the sample mean and standard deviation of the vehicle weights. From the mean, μ = 1,551 kg (3,413.3 lb), and the standard deviation, σ = 483 kg (1,063.6 lb), the values of the lognormal distribution parameters $\lambda = E(\ln x)$ and $\varsigma = \sigma_{\ln x}$ are determined from the first two moments. The validity of the assumption of the lognormal distribution can then be checked against the survey sample distribution as shown in Fig. 2. The fit is reasonably good, particularly at the tail. Since the interest is focused on the extreme load effect produced by the vehicle weights, the fit at the tail should carry more weight. In other words, it is the heavier vehicles [say, those that weigh more than 1,818 kg (4,000 lb)] that most likely will produce the governing load effects in the building for the determination of the design load, and the lognormal model gives an accurate distribution of these vehicles. For this reason, commonly used statistical tests that put equal weight on all values of the variable would not be productive, and therefore are not performed. However, to ensure that the lognormal model is the most appropriate distribution, in Fig. 2, the survey sample distribution is also compared with other commonly used live load distributions, namely, normal and gamma distributions with the same first two moments. It can be seen that the lognormal distribution is indeed the best among the three models.

Column Axial Load

A schematic representation of the stall layout in a bay of a typical parking garage is shown in Fig. 3. The layout is presented in such a manner as to facilitate the computation of the influence coefficients for the column. Note that the parking bays considered in this section only allow one-way traffic. Parking bays that allow two-way traffic will be considered in a later section. The concept of influence area will be used for the analysis of the axial load supported by each column. The load position with respect to the column influences the extent of loading supported by the column. Intuitively, the closer the load is to the column, the more significant the effect that it has on the column, and vice versa. An influence surface shows the effect of a unit load anywhere on the slab on the column axial loading or beam midspan moment—the two cases that will be considered in this study (McGuire and Cornell 1974; Ellingwood and Culver 1977).

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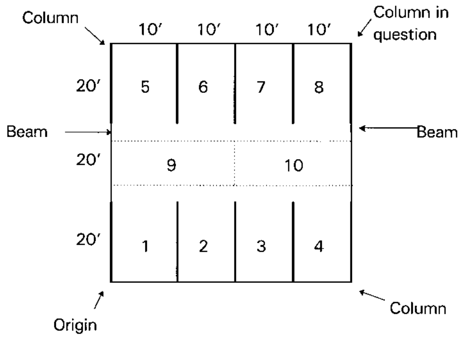

FIG. 3. Stall Layout and Vehicle Position for Evaluation of Column Axial Force and Midspan Beam Moment (1 ft = 0.3048 m)

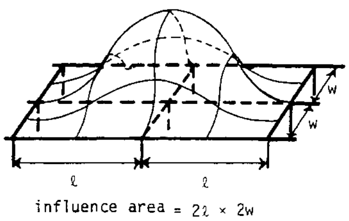

FIG. 4. Influence Surface for Column Axial Loading

For a column, the influence surface of the load on its axial loading is shown in Fig. 4 (McGuire and Cornell 1974; Corotis and Doshi 1977; Chalk and Corotis 1980). Note that the influence area for the axial loading on a column is four times the conventional tributary area used in design analysis. An equation that describes the influence surface shown above is given by (Corotis 1972)

$$C(x, y) = (3x^2 - 2x^3)(3y^2 - 2y^3) \quad (2)$$

where C = influence coefficient; and x and y = normalized spatial variables ranging from zero to one. From Fig. 4, we can obtain the normalized spatial variables of x and y by dividing the coordinates of each stall by the bay length or width. Approximating the vehicle load by a concentrated force, the locations of the force in terms of the normalized coordinate pairs and the corresponding influence coefficients are as follows:

- Stall 1: (0.125, 0.1667), C_1 = 0.00318
- Stall 2: (0.375, 0.1667), C_2 = 0.02344
- Stall 3: (0.625, 0.1667), C_3 = 0.05064
- Stall 4: (0.875, 0.1667), C_4 = 0.07089
- Stall 5: (0.125, 0.8333), C_5 = 0.03979
- Stall 6: (0.375, 0.8333), C_6 = 0.29297
- Stall 7: (0.625, 0.8333), C_7 = 0.63296
- Stall 8: (0.875, 0.8333), C_8 = 0.88614

Hence, for the bay shown in Fig. 3, the total axial load supported by one column is given by the following formula:

$$F = C_1W_1 + C_2W_2 + C_3W_3 + C_4W_4 + C_5W_5 + C_6W_6 + C_7W_7 + C_8W_8 \quad (3)$$

where F = axial loading on one column due to the load on 1/4 of the influence area; and W_i = random variables of vehicle weights, assumed to be statistically independent and to have the same mean and standard deviation. Thus, the mean value and standard deviation of F are

$$E(F) = (C_1 + C_2 + C_3 + C_4 + C_5 + C_6 + C_7 + C_8)E(W) = 3{,}102 \text{ kg } (6{,}826 \text{ lb}) \quad (4)$$

$$\sigma_F = \sqrt{C_1^2 + C_2^2 + C_3^2 + C_4^2 + C_5^2 + C_7^2 + C_8^2}\ \sigma_W = 547 \text{ kg } (1{,}205 \text{ lb}) \quad (5)$$

A linear function of the lognormal distribution such as the axial load is no longer lognormal. According to the central-limit theorem, the summation of a large number of random variables will approach a normal distribution. Since only eight random variables are in the computation of the axial load and the contribution of two stalls (7 and 8) dominates, the distribution would be somewhere in-between. A lognormal distribution would be a good conservative assumption in this case. In this study, both the normal distribution and the lognormal distribution will be used, and a comparison of their respective extreme values will be made. In general, when the coefficient of variation is small, such as in the case for the axial load (δ = 0.19), the difference between the normal and lognormal distribution will be small.

Extreme Column Axial Loading

The mean and standard deviation of the column axial force obtained above are for one ''realization'' of a fully parked bay. During the garage's lifetime, there will be a large number of such realizations. For a typical commercial parking garage, it is reasonable to assume that the axial load due to such realizations varies (statistically) independently from day to day. We are interested in the statistics of the lifetime maximum load effect. For this purpose, we can use the asymptotic distributions of the extreme values (Ang and Tang 1984). The distribution for the load effect will be assumed to be either normal or lognormal, as indicated in the previous section.

Extreme Axial Loading Based on Normal Distribution

Assuming that the central-limit theorem is valid, F will approach a normal distribution and $E(F)$ = 3,102 kg (6,826 lb) and σ_F = 547 kg (1,205 lb). F_n, the extreme values of F for a sample size of n realizations, can be shown to approach the type I asymptotic distribution (Ang and Tang 1984). $E(F_n)$ is obtained as a function of n and is shown in Fig. 5. We see that the value of $E(F_n)$ becomes less sensitive to the sample size for large n. For a design life of 30 years, $n \approx 11{,}000$. At this value of n, F_n has an expected value of 5,271 kg (11,598 lb) and a coefficient of variation of only 3.37%. We can conclude that the maximum column axial load due to the load in a bay area (i.e., 1/4 of the influence area for the column) will be close to 5,273 kg (11,600 lb).

Extreme Axial Loading Based on Lognormal Distribution

As explained above, the lognormal distribution may be a more appropriate model for F due to the relatively small number of vehicles in a typical bay area and the dominance of several vehicles on the load effect. The extreme value of the lognormal random variable approaches a type II asymptotic distribution (Ang and Tang 1984). $E(F_n)$ as function of n is obtained and is shown in Fig. 5. Considering the relative insensitivity of the value of $E(F_n)$ to the sample size for large n, we conclude that the 30-year maximum column axial load has a mean value of 6,080 kg (13,376 lb) (for n = 11,000) and a very small coefficient of variation. From the calculations above, it is seen that the assumption of an initial lognormal distribution gives an increase of about 15% in the mean value of the extreme axial loads.

Now, consider a separate scenario where there are vehicles occupying the aisle between the parking stalls, as shown in Fig. 3. The occurrence of a fully occupied aisle will generally be a much less frequent event over the structural life span. Again we assume a sample size of n = 11,000 for one realization per day for 30 years, a rather conservative assumption. A similar analysis using the lognormal distribution results in a maximum axial load of 6,925 kg (15,236 lb), which is higher

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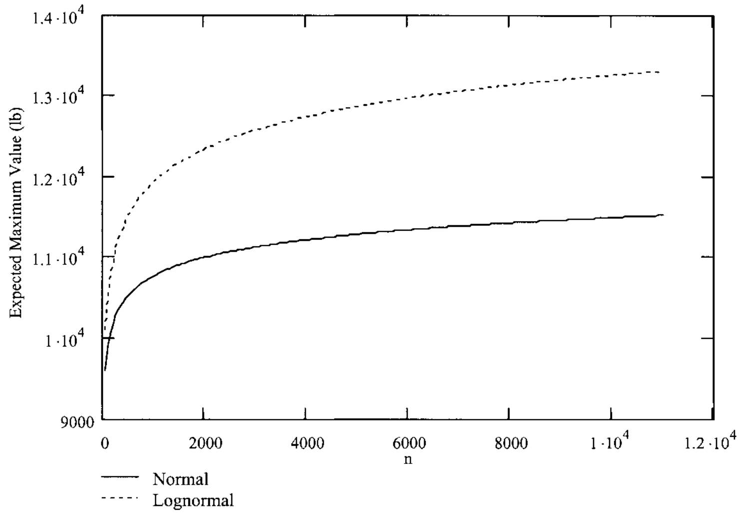

FIG. 5. Mean Maximum Column Axial Force As Function of Number of Realizations (1 lb = 0.454 kg)

than the previous estimate of maximum axial load of 6,080 kg (13,376 lb).

Assuming a uniform load of 2.39 kN/m^2 (50 psf) the expected axial load on each column due to the load in this bay area (i.e., 1/4 of the influence area) is equal to 50 × 40 × 60/4 = 13,636 kg (30,000 lb). The axial load on each column due to a design load of 2.39 kN/m^2 50 psf is about twice the value of the maximum axial load that we expect the column to experience over the garage's lifetime. If the axial load on each column is assumed to be 6,925 kg (15,236 lb), the EUDL is calculated to be 1.22 kN/m^2 (25.4 psf).

Beam Midspan Moment

Referring again to Fig. 3, for a beam, the influence surface for the midspan moment is shown in Fig. 6 (McGuire and Cornell 1974). Parking garages that allow two-way traffic flow will be considered in a later section. Note that the influence area for the midspan moment on the beam is twice the conventional tributary area used in design. The influence coefficient can be described by a polynomial along the y-axis (perpendicular to the beam) and an exponential function along the x-axis (parallel to the beam), as follows:

$$C(x, y) = e^{-0.116x}(1 + 0.05y)(1 - 0.025y)^2 \qquad (6)$$

where C = influence coefficient; and x and y = spatial variables. It is seen that the shape of the influence surface is symmetrical about both the x-axis and the y-axis, with the origin at the center of the beam. The coordinate pairs and their corresponding influence coefficients are

- Stall 1: (20, 35), $C_1 = 0.00422$
- Stall 2: (20, 25), $C_2 = 0.03109$
- Stall 3: (20, 15), $C_3 = 0.06718$
- Stall 4: (20, 5), $C_4 = 0.09405$

The midspan moment due to a load at (x, y) from the center of the beam can be computed by

$$M(x, y) = M(0, 0) \times C(x, y)$$

where $M(0, 0)$ = midspan moment when the load is located at the center of the beam. Hence, the midspan moment due to vehicles 1–8 in Fig. 3 (no vehicles in the aisle) can be calculated as follows:

$$M = C_1(7.5W_1) + C_2(7.5W_2) + C_3(7.5W_3) + C_4(7.5W_4) + C_5(7.5W_5) + C_6(7.5W_6) + C_7(7.5W_7) + C_8(7.5W_8) \qquad (7)$$

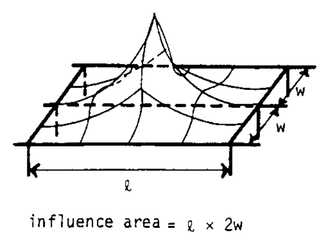

FIG. 6. Influence Surface for Beam Midspan Moment

and

$$E(M) = 7.5(2C_1 + 2C_2 + 2C_3 + 2C_4)E(W) = 13.8 \text{ kN}\cdot\text{m } (10{,}176 \text{ lb}\cdot\text{ft}) \qquad (8)$$

$$\sigma_M = 7.5\sqrt{2C_1^2 + 2C_2^2 + 2C_3^2 + 2C_4^2}\,\sigma_W = 1.83 \text{ kN}\cdot\text{m } (1{,}352 \text{ lb}\cdot\text{ft}) \qquad (9)$$

Extreme Midspan Moment

Again, the analysis will be based on both a normal distribution and a lognormal distribution, and a comparison of their respective extreme values will be made. Assuming first an initial normal distribution for M, with the mean and standard deviation given by (8) and (9), M_n, the extreme values of M for a sample size of n realizations, will approach the type I asymptotic distribution. The expected value of M_n as a function of the sample size n is shown in Fig. 7. A reasonable estimate of the lifetime maximum midspan beam moment is 20.9 kN·m (15,440 lb·ft) (for n = 11,000 for a duration of 30 years). Assuming then an initial lognormal distribution for M, we can obtain the expected value $E(M_n)$ as a function of n, as shown in Fig. 7. A good estimate of the lifetime maximum midspan moment is 22.9 kN·m (16,900 lb·ft) (for t = 30 years and n = 11,000). As in the previous case, the lognormal distribution gives a slightly higher result.

For the scenario where there are vehicles (numbers 9 and 10 in Fig. 3) occupying the aisle between the fully parked stalls, a similar analysis using the lognormal distribution results in a maximum midspan moment of 101 kN·m (74,900 lb·ft) for n = 11,000 (30 years). This value is much larger than that obtained without the vehicles in the aisle. It is attributed to the proximity of the vehicles to the center of the beam, which results in a much higher midspan moment (see the midspan moment influence surface in Fig. 6). Hence, a midspan moment of 101 kN·m (74,900 lb·ft) will be used.

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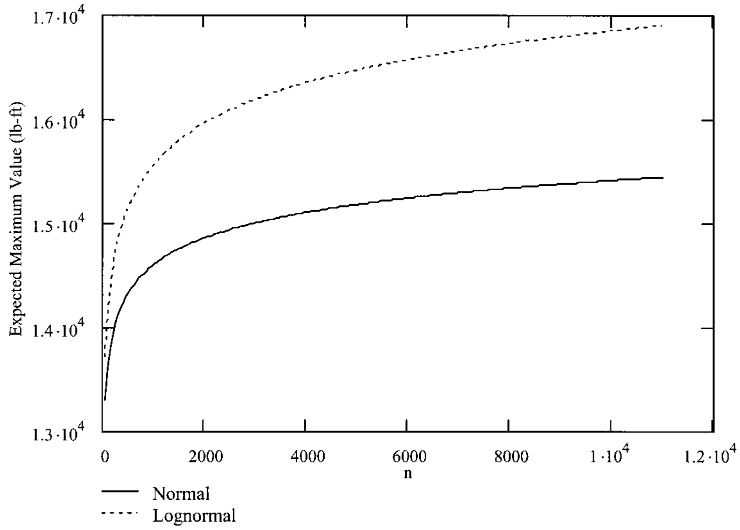

FIG. 7. Mean Maximum Midspan Bending Moment As Function of Number of Realizations (1 lb/ft = 1.26 N/m)

Assuming a uniform load of 2.39 kN/m^2 (50 psf) over the bay area, the midspan moment on each beam can be calculated from the influence surface function as follows:

$$M = 4 \times 50 \times 7.5 \times \int_0^{30} \int_0^{40} C(x, y)\, dx\, dy$$

$$= 350 \text{ kN}\cdot\text{m } (258{,}620 \text{ lb}\cdot\text{ft}) \quad (10)$$

It is seen that the midspan moment due to a design load of 2.39 kN/m^2 (50 psf) is more than three times the value of the maximum midspan moment that we expect the beam to experience over its life span. If the midspan moment on each beam is assumed to be 101 kN·m (74,900 lb·ft), using the reverse of the process described above, the equivalent uniformly distributed load can be calculated to be 0.695 kN/m^2 (14.5 psf).

Parking Garage with Two-Way Traffic

For a layout of a typical parking bay that allows two-way traffic, based on conservative assumptions of a sample size of 11,000 and a lognormal distribution for both the axial loading and the midspan moment, the maximum axial load is determined to be 7,856 kg (17,285 lb). The corresponding equivalent uniformly distributed load is calculated to be 1.27 kN/m^2 (26.6 psf). The EUDL for the maximum midspan moment is much less.

DYNAMIC EFFECTS OF MOVING VEHICLES

A study of moving vehicle loads on the garage decks is also carried out to evaluate the effects of dynamic floor slab oscillation. Literature surveys indicated that a common construction method for parking garages is precast, prestressed concrete or cast-in-place posttensioned concrete beams and slabs (Tyson 1999). For the dynamic response analysis, we use a floor slab model of a flat, homogeneous, rectangular plate, 18.3 m (60 ft) long by 12.2 m (40 ft) wide, supported by beams. Other commonly used methods such as steel floor joists would likely result in stiffer slabs and less dynamic oscillation. We use a Young's modulus of 3.1 10^7 kN/m^2 (4,500 ksi) and a Poisson's ratio of 0.15. The density of the concrete is assumed to be 2,407 kg/m^2 (150 lb/ft^3) with a thickness of 0.152 m (6 in.). The dynamic study of vehicular loads will be done in three steps. The first step is the modeling of the movement of a single vehicle over a typical floor slab. The effects of the velocity of the vehicle and the slab dimension on the dynamic amplification will be examined. The second step is the modeling of the movements of several vehicles over a floor slab, and the effects of the number of vehicles on the dynamic amplification will be investigated. The third step is the analysis of the combined effect of both the static loads and the dynamic loads due to moving vehicles over the floor slab. An equivalent uniformly distributed load will then be determined for the lifetime maximum load effect. Study of the dynamic amplification factor (DAF) for the floor slabs requires proper modeling of the boundary conditions of the floor slab in a given bay. Since it is generally difficult to determine the actual boundary conditions, both a simply supported plate and a fully clamped plate will be considered in the following. The latter is a more realistic assumption for a continuous plate.

Floor Response under Single Moving Vehicle

For a simply supported plate on all four edges, the mode shapes under free vibration can be expressed in terms of simple sinusoidal functions. Consider first the contribution of the first mode only, whose natural frequency is determined to be 2.43 cycles/s. The dynamic response of the slab under a single moving concentrated load depends on the speed of the moving load and the span length. We focus on the dynamic response at the center of the slab, which would be most severe and can be used as a good measure of dynamic amplification. For the range of speed and the size of the slab, the higher mode contribution compared with that of the first mode will be examined. It is assumed in this study that the vehicle is a concentrated moving load along the centerline of the slab. This is a conservative assumption, since in reality the four wheel loads will each have only a fraction of the total weight and will not be at the centerline of the plate, and the dynamic forces due to each wheel load will be slightly out of phase. The total dynamic effect would therefore be less severe. The displacement at the center of the slab due to the first mode as a function of time elapsed after the vehicle moves on the plate and before the vehicle leaves the plate is given by

$$y(t) = \frac{4P_0}{M}\frac{1}{\omega_d}\int_0^t e^{-\varsigma\omega_n(t-\tau)}\sin[\omega_d(t-\tau)]\sin(\pi Vt/a)\,d\tau$$

$$\text{for } t < a/V \quad (11)$$

where a = width of plate; P_0 = weight of car; M = mass of plate = ρabh (where ρ = density of plate) = 81,800 kg (5,590 slugs); V = velocity of vehicle; ω_n = undamped natural fre-

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quency of plate vibration; and $\omega_d = \omega_n\sqrt{1 - \zeta^2}$ = damped natural frequency. The displacement due to free vibration after the vehicle leaves the plate is given by (11), with the upper limit of integration changed to a/V.

For a slab with four edges clamped, the free vibration problem is considerably more complex. The first mode natural frequency is determined to be 4.61 cps based on Magrab (1979). Using an approximate cosine mode shape function, the displacement at the center of the slab as a function of time when the vehicle is still on the plate is given by

$$y(t) = \frac{8P_0}{M}\frac{1}{\omega_d}\int_0^t e^{-\zeta\omega_n(t-\tau)}\sin[\omega_d(t-\tau)]\cos\left(\frac{2\pi Vt}{a} - 1\right)d\tau$$

$$\text{for } t < a/V \qquad (12)$$

Again, after the vehicle leaves the plate, the upper limit of integration is changed to a/V. The displacement time histories under vehicles moving at different speeds are calculated according to (11) and (12), in which a damping ratio of 2% is assumed. The displacement (ft) according to (12) as a function of time elapsed after a vehicle of 1,551 kg (3,413.3 lb) moves on a clamped slab is shown in Fig. 8 for three vehicle speeds. The static deflection under the weight of the vehicle at the center of the slab is 0.00166 m (0.00547 ft). It is seen that at low speed, the response is quasi-static with little dynamic oscillation. The dynamic amplification increases with vehicle speed and becomes significant only at an extremely high speed, which is unlikely to occur in a parking garage.

The dynamic amplification factor, defined as the ratio of the maximum dynamic response to the static response, of a vehicle

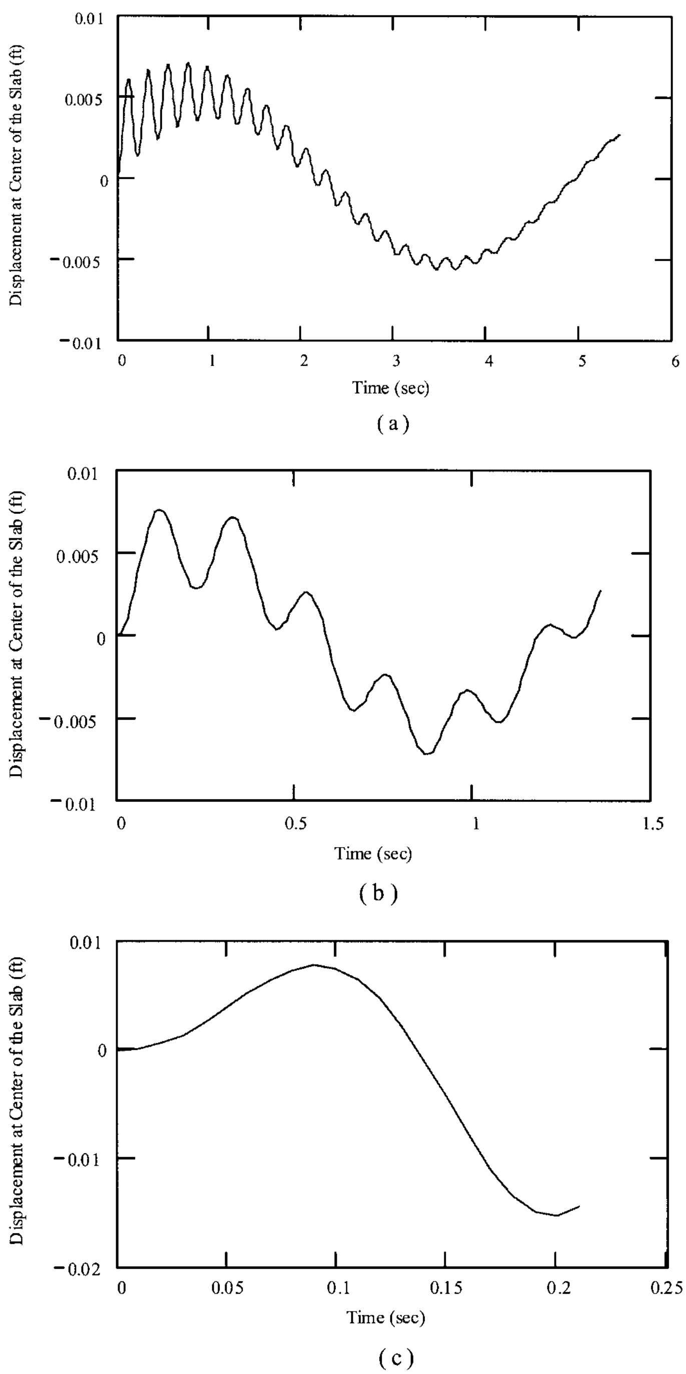

FIG. 8. Displacement at Center of Slab for: (a) V = 8 km/h (5 mph); (b) V = 32 km/h (20 mph); (c) V = 202 km/h (126 mph) (1 ft = 0.3048 m)

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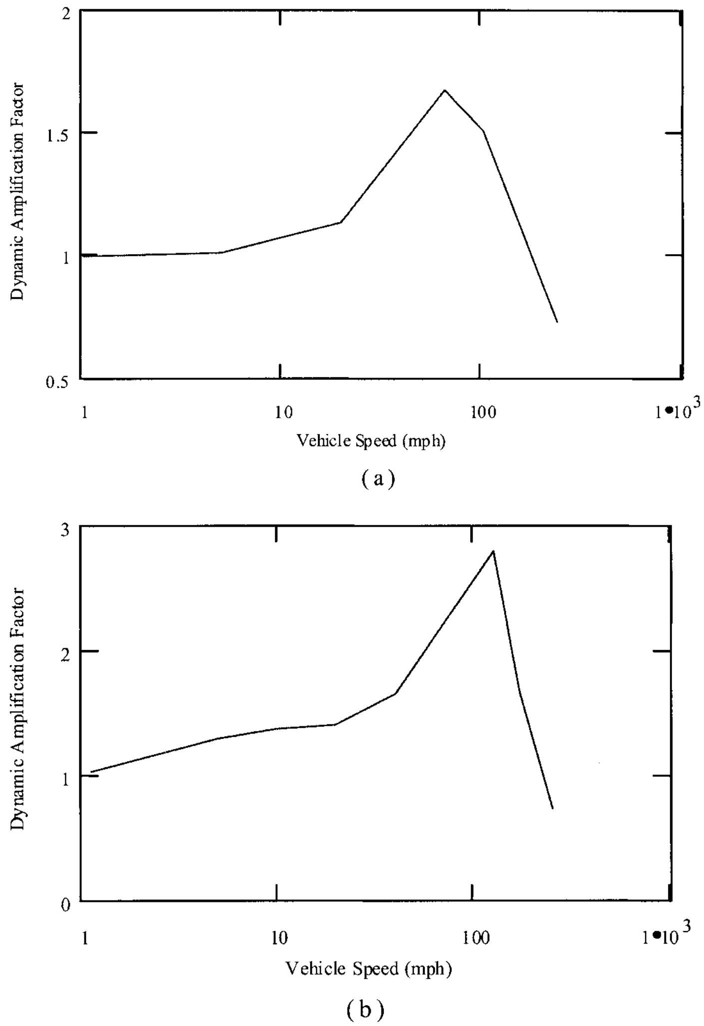

FIG. 9. Dynamic Amplification Factor As Function of Vehicle Speed: (a) Simply Supported Slab; (b) Clamped Slab (1 mph = 1.6 km/h)

parked at the center of the slab is shown in Fig. 9 for both simply supported and clamped boundary conditions. It reaches a maximum of 1.7 at 106 km/h (66 mph) for the simply supported slab and 2.8 at 202 km/h (126 mph) for the clamped slab when the duration of the moving load on the slab coincides with the natural period of the slab vibration. In the normal range of vehicle speeds in parking garages, the amplification factor is quite small. For example, it is less than 1.33 for vehicle speeds less than 10 mph and less than 1.5 for speeds less than 80 km/h (50 mph).

The actual dynamic amplification that is experienced by a continuous concrete slab is probably much closer to that of a fully clamped than a simply supported plate. Hence, the fully clamped scenario will be considered in the rest of the paper. The dynamic amplification factor is independent of the weight of the vehicle. It is seen from the above parametric study that the dynamic amplification is highest when the duration of the vehicle on the plate coincides with the natural period of the plate. Reducing the width of the plate reduces the natural period, but also reduces the duration of the vehicle on the slab for a given speed. These two effects tend to cancel each other in terms of dynamic amplification. The effect of the width of the plate on the dynamic amplification would therefore be small. For normal traffic conditions, one expects the vehicular speed to be much less than the values that achieve maximum dynamic amplification. A conservative factor of 2.0, therefore, is used in this study to convert the dynamic loads to equivalent static loads for a single moving vehicle.

Effect of Higher Modes

The approximation of using only the fundamental mode in the above analysis is examined by also considering the next few higher modes contributing to the displacement at the center of the slab. For the clamped slab, the next mode has a natural frequency of 11.35 cycles/s. The contribution of this mode to the total displacement for the case of V = 32 km/h (20 mph) is about one order of magnitude smaller. The peak displacement increases only 13% by adding this mode. The rest of the contributing higher modes would add approximately 6%, 2%, etc. to the peak response, which would be adequately accounted for in the conservative estimate of the dynamic amplification factor of 2.0.

Floor Response under Several Moving Vehicles

The combined effect of the movement of several vehicles across the concrete slab can be obtained by superposing the slab displacement due to each vehicle under the assumption that the plate response is linear. It can be seen that the maximum dynamic amplification occurs in a situation where there are only two vehicles, since the width of the bay is only sufficient to accommodate two vehicles.

To evaluate the response, we assumed two vehicles moving across the slab at 8 km/h (5 mph) (a reasonable speed, considering the fact that the vehicles are moving one after the other) at a spacing of 2.4 m (8 ft) apart. The plate response equations in the previous sections are used, and the slab displacements due to two vehicles are superposed to obtain the slab response. From Fig. 8(a), one can see that at such low speeds, the slab response is quasi-static, with very little dynamic amplification. In other words, the maximum deflection at the center of the slab would not be too much different from that due to two stationary vehicles waiting in the lane. For example, according to the above scenario of two vehicles in tandem, the lag time for the second vehicle is approximately 3–4 s, depending on the length of the vehicles. It is seen by superposition of the time histories that the maximum response would probably be even smaller than that due to a single vehicle because of cancellation. Other scenarios may produce slightly different results. The overall dynamic amplification, however, would be small for multiple moving vehicles. The maximum static deflection can be computed using the midspan deflection influence coefficients for vehicles not at the center of the plate. The influence coefficients for the deflections are given by the following equation, using normalized coordinates (McGuire and Cornell 1974):

$$I(x) = 3x^2 - 2x^3 \quad \text{for } 0 < x < 1 \tag{13}$$

where x is normalized by half of the span length. Assuming that two stationary vehicles are in the aisle at quarter-span points of the concrete slab, the maximum static deflection due to the two vehicles is about the same as that of a single vehicle in the center.

Dynamic Effects on Column Axial Load and Beam Bending Moment

Consider the layout of the parking garage in Fig. 3. The effects of dynamic amplification due to moving vehicles can be taken into account by using an equivalent static load that produces the same load effect. The effects of moving vehicles can be incorporated by multiplying the static load effects by the dynamic amplification factor obtained above. Consider first the case of a single vehicle moving across the slab. It has been shown earlier that the dynamic amplification factor is independent of the weight of the vehicle, and only dependent on the velocity of the vehicle. The DAF is also shown to be less than 2. The total load supported by one column is given by

$$F = C_1W_1 + C_2W_2 + C_3W_3 + C_4W_4 + C_5W_5 + C_6W_6 + C_7W_7 + C_8W_8 + C_mW_m \tag{14}$$

where F = axial loading on one column due to the load on 1/4 of the influence area; W_i = statistically independent random variable for the weight of each parked vehicle; C_I = column force influence coefficient; W_m = weight of the moving vehicle; and C_m = effective static load coefficient due to the moving vehicle = influence coefficient × DAF. Similarly, the midspan moment M sustained by one beam is given by

$$M = C_1(7.5W_1) + C_2(7.5W_2) + C_3(7.5W_3) + C_4(7.5W_4) + C_1(7.5W_5) + C_2(7.5W_6) + C_3(7.5W_7) + C_8(7.5W_8) + C_m(7.5W_m)$$

Since the DAF for a single moving vehicle is less than 2, the dynamic effect would not be more than that, due to the static effect of four vehicles waiting in the aisles, two in each direction, which has been considered before.

For the case of two vehicles moving across the slab, it has been shown that the resulting dynamic amplification would be small. Therefore, the resultant load effects on the columns and beams would again be less than in the case of four vehicles waiting in the aisles for two-way traffic parking, the case considered before. Finally, when two vehicles are moving in opposite directions in a two-way traffic situation, the dynamic amplification factor would be less than that of a single vehicle, since the peak dynamic deflections would be much less likely to occur at the same time. It would be reasonable to conclude that dynamic amplification due to moving vehicles would not cause EUDL to be more than the case of fully parked two-way traffic bays with vehicles waiting in the aisles, which has been estimated to be 1.27 kN/m^2 (26.6 psi).

Dynamic Impact on Ramp

When a vehicle moves onto a ramp with a grade, there is an impact force on the ramp that should be added to the static force. The impact force depends on the ramp grade, the vehicle velocity, and vehicle dynamic characteristics. The exact solution to this problem is rather complicated, since one has to consider the impact effect of each of the four wheels and the six degrees of freedom of the vehicle dynamic motion. In the following, to estimate approximately the impact effect, the problem is idealized as a single spring-mass-damper system on a single wheel. The vertical motion of the vehicle can be described by a single degree of freedom system under an impact type of excitation. From dynamics, the maximum deflection of the mass under an impact type of force is equal to the deflection under a static force multiplied by an impact magnification factor. One can multiply this maximum deflection by the spring constant to arrive at the maximum force exerted on the ramp. The force magnitude, however, is a function of the vehicle weight, velocity, grade, the rate of grade change, and the size of the car wheel and tire, making exact evaluation difficult. As an approximate estimate, consider the following system parameters:

- Vehicle weight = 1.818 kg (4,000 lb) (m = 124.2 slug)
- Spring constant, k = 29,230 kg/m (19,600 lb/ft)
- Vehicle velocity, V = 5 mph (2.23 m/s[7.33 ft/s])
- Ramp grade = 6°

The natural frequency of the vehicle is 2 cycles/s. Assuming a short (0.91 m [3 ft]) but smooth (parabolic) transition from the deck to the ramp

$$g(x) = 0.0348\, x^2 = 0.348(Vt)^2 \quad \text{for } 0 < x < 0.91 \text{ m (3 ft)}$$

The equation of motion of the vehicle in terms of the vertical relative motion $Y(t)$ to the ramp surface is given by

$$m\ddot{y} + c\dot{y} + ky = -m\ddot{g} \tag{15}$$

The force on the right side = 124.2 × 0.348 × 2 × 7.33 = 288 kg (633.6 lb), which lasts for 3/7.33 = 0.409 s. The solution under the zero initial condition is obtained as

$$Y(t) = \frac{m\ddot{g}}{k}\left[1 - \frac{e^{-\zeta\omega t}}{\sqrt{1-\varsigma^2}}\cos(\sqrt{1-\varsigma^2}\,\omega t - \psi)\right] \quad \text{for } t < 0.409 \text{ s} \tag{16}$$

where ζ = damping ratio; ω = circular natural frequency; and $\psi = \tan^{-1}(\zeta/\sqrt{1-\zeta^2})$.The damping ratio for vehicles is normally around 40%. The maximum displacement can be calculated from the above equation and the impact magnification factor is very close to 1.0, a well-known result for systems with large damping (Chopra 1995). The maximum impact force is, therefore, 288 kg (633.6 lb), and the impact factor in terms of the vehicle weight would be 1 + 633.6/4,000 = 1.16. As can be seen, this factor increases with vehicle velocity and ramp grade. The same vehicle moving at a speed of 16 km/h (10 mph) on a ramp with a grade of 10% (maximum allowed for the ramp) would have an impact factor of 1.54.

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If the transition region is much shorter, with the transition time less than half of the vehicle natural period, the impact would be of the impulse type, and the vehicle response time history (with critical damping) is

$$Y(t) = \frac{I}{m} te^{-\omega t} \quad (17)$$

where I = impulse = $m\Delta V$; and $\Delta V = 7.33 \times \sin 6° = 0.22$ m/s (0.73 ft/s), the change of vehicle vertical velocity from zero after the transition. The maximum response can be found from the above equation to be 0.013 m (0.042 ft). The impact factor is

$$1 + 0.042 \times 19{,}600/4{,}000 = 1.2$$

which is slightly greater than the previous case of 1.16. Again for V = 16 km/h (10 mph) and a grade of 10°, the factor increases to 1.59.

The assumption of a single mass for the vehicle, however, is conservative, since in reality the vehicle motion has at least two degrees of freedom when both the front and the rear wheels are considered, and the impact force would be somewhat less than the above values. It can be concluded from the above that a value of 1.5 would be a good conservative estimate of the dynamic impact factor on ramps.

OTHER CONSIDERATIONS

Variation of Stall Size

The stall width can be as narrow as 2.4 m (8 ft). In the above typical bay, there can be 10 stalls with parked vehicles. The load effect would obviously be higher. An analysis parallel to the above was carried out for each case. The governing case of column axial load for two-way traffic with vehicles in the aisles (Fig. 10) gives a 30-year maximum value of 8,876 kg (19,528 lb) (EUDL of 1.44 kN/m^2 30 psf) if a lognormal distribution is used and 7,882 kg (17,340 lb) (EUDL of 1.27 kN/m^2 [26.64 psf]) if a normal distribution is used for the load effect. In passing, it is pointed out that some commercial valet parking garages may have more densely parked cars in a given bay, but with cars parked only in half of the aisle. The resulting load effects would not be about the same as the above values.

Variation of Bay Size

It is noted that the above models are for typical parking garages. There is variation of bay size and stall layout (such as layouts with different angles). An angled stall layout increases the stall width and slightly reduces the length of the bay. The net effect on the EUDL would be minimal. In the following, the bay width is varied to see the sensitivity of the design load to such variation. It is not uncommon to have a bay size of 9.1 m (30 ft) by 18.2 m (60 ft) with three stalls on each side, and a precast deck with beams spaced 9.1 m (30 ft) apart. In these designs, the bays are smaller. For a 9.1 m (30 ft) by 18.2 m (60 ft) bay allowing three stalls on each side, the governing case of a two-way traffic bay with vehicles in the aisles is considered. In this case, the number of vehicles in the bay is reduced; however, the bay area is also reduced proportionally. It is therefore expected that the EUDL would not be sensitive to the bay size variation. To verify this observation, the 30-year maximum column axial load is calculated. It is found to be 5,940 kg (13,069 lb). The corresponding EUDL is therefore 13,069/450 = 1.39 kN/m^2 (29.0 psf), which is only slightly greater than the 1.27 kN/m^2 (26.6 psf) for the previous case of a larger bay with four stalls on each side. The EUDL for the 30-year maximum midspan beam moment is much less than 1.44 kN/m^2 (29.0 psf).

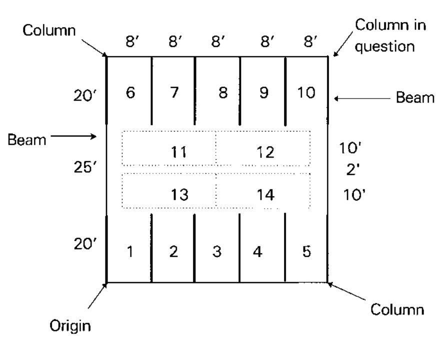

FIG. 10. Vehicle and Stall Layout for 10-Stall Bay (1 ft = 0.3048 m)

Passenger and Cargo Loading Factors

It is difficult to ascertain the actual distribution of passenger and cargo loading in vehicles. Literature surveys also demonstrate a lack of information in this area. As a result, an approximate analysis is performed to estimate the effects that the increased loads have on the equivalent uniformly distributed load.

The passenger and cargo load is assumed to have a mean value of about 17% of the actual weight of the car, which corresponds approximately to two passengers plus 113 kg (250 lb) of cargo load, including gas. The coefficient of variation is assumed to be 33% (standard deviation equal to one-third of the mean value). The total weight of the vehicle can be represented by a new random variable, given by

$$\text{total weight of the vehicle, } W_T = (LF) \times W$$

where W_T = total weight of the vehicle, inclusive of the passenger and cargo load; LF = loading factor, a random variable; and W = weight of the empty vehicle.

From the estimation of the passenger and cargo load above, the LF has a mean value of 1.07 and a standard deviation of 0.016 for parked vehicles (with no passengers), and 1.17 and 0.05 for moving vehicles (with passengers). Assuming also that the loading factor has a lognormal distribution, LF has $\lambda = 0.0675$ and $\zeta = 0.016$ for parked vehicles, and $\lambda = 0.155$ and $\zeta = 0.0435$ for moving vehicles; we can then determine the statistics for W_T to be $\lambda = 8.101 + 0.0675 = 8.168$ and $\zeta = \sqrt{0.301^2 + 0.016^2} = 0.301$ for parked vehicles, and $\lambda = 8.101 + 0.155 = 8.255$ and $\zeta = \sqrt{0.301^2 + 0.0435^2} = 0.304$ for moving vehicles.

Using these total vehicle weight statistics, we calculated the governing case of the maximum axial loading supported by the column for a bay with two-way traffic and moving vehicles in both directions. The EUDL for the 30-year (n = 11,000) maximum load effect is found to be 1.34 kN/m^2 (28.1 psf) for an eight-stall bay and 1.53 kN/m^2 (32.0 psf) for a 10-stall bay, using a normal distribution. A lognormal distribution assumption would yield, correspondingly, 1.48 kN/m^2 (30.8 psf) for an eight-stall bay and 1.67 kN/m^2 (34.8 psf) for a 10-stall bay.

Future Impact of Very Heavy Vehicles

In recent years, some sport-utility vehicles have gotten heavier, and some of them exceed 4,545 kg (10,000 lb). However, these heavy vehicles still represent a small percentage of the database (e.g., only two 3,909 kg (8,600 lb) Chevy Suburbans out of 519 vehicles in the load surveys). The effect of such vehicles could be important due to regional variation and could become more pronounced in the future.

SUMMARY AND RECOMMENDATIONS

Load surveys of vehicle weights were conducted in nine commercial parking garages. Statistical analyses of the maxi-

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mum load effects on beams and columns due to vehicle loads over the lifetime of the garages were carried out using the survey results. Dynamic effects on the deck due to vehicle motions and on the ramp due to impact were investigated. Effects of bay size variation, and passenger and cargo loading factors were also considered. In the analyses, when data were unavailable, reasonable and conservative assumptions regarding vehicle parking and loading patterns were made. The results are summarized as follows:

1. The EUDLs that would produce the 30-year maximum column axial force and midspan beam bending moment are conservatively estimated to be 1.48 kN/m^2 (30.8 psf) for an eight-stall bay and 1.67 kN/m^2 (34.8 psf) for a 10-stall bay of a typical size of 12.2 m (40 ft) by 18.3 m (60 ft).
2. The EUDL is not sensitive to bay-size variation.
3. The dynamic effect (defection) due to moving vehicles on the deck is mainly a function of the vehicle speed, and is less than two times that of the static defection for a single moving vehicle. The dynamic amplification is reduced to close to unity when vehicles are moving in tandem. The net effects of moving vehicles are found to be not higher than in the case of the driving aisles being fully occupied by stationary vehicles waiting in line.
4. The impact force on the ramps is conservatively estimated to be 1.5 times a single vehicle weight.

Based on the above results, we conclude that the current design load of 2.39 kN/m^2 (50 psf) in ASCE-7-95 is conservative compared with the estimated 30-year maximum EUDL of 1.67 kN/m^2 (34.8 psf). In view of the possible impact of very heavy sport-utility vehicles, however, a design load of 1.92 kN/m^2 (40 psf) is recommended, with no allowance for reduction according to bay area. It represents a 20% reduction from the current ASCE-7-95 value, but is still much higher than the 1.44 kN/m^2 (30 psf) when an area-based reduction is applied for large bays, as allowed in most standards. An impact load of 1.5 times a single vehicle weight may be used for ramps. The recommendations, however, apply only to regular commercial parking garages and not other special-purpose garages where vehicles may be more densely parked or where the same type of heavy vehicles are parked, causing higher load effects.

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APPENDIX. REFERENCES

Ang, H. S., and Tang, H. (1984). *Probability concepts in engineering planning and design: Volume 2—Decision, risk and reliability*, Wiley, New York.

ASCE. (1996). ''Minimum design loads for buildings and other structures.'' *ASCE-7-95*, Reston, Va.

Automotive news: Market data book. (1998). Crain Communications, Detroit.

Chalk, P. L., and Corotis, R. B. (1980). ''Probability model for design live loads.'' *J. Struct. Div.*, ASCE, 106(10), 2017–2033.

Chopra, A. K. (1995). *Dynamics of structures: Theory and applications to earthquake engineering*, Prentice-Hall, Englewood Cliffs, N.J.

Corotis, R. B. (1972). ''Statistical analysis of live load in column design.'' *J. Struct. Div.*, ASCE, 98(8), 1803–1815.

Corotis, R. B., and Doshi, V. A. (1977). ''Probability models for live-load survey results.'' *J. Struct. Div.*, ASCE, 103(6), 1257–1274.

Ellingwood, B. R., and Culver, G. C. (1977). ''Analysis of live loads in office buildings.'' *J. Struct. Div.*, ASCE, 103(8), 1551–1560.

International Conference of Building Officials (ICBO). (1997). *Uniform building code*, Whittier, Calif.

Magrab, E. B. (1979). *Vibration of elastic structural members*, Sijthoff & Noordhoff.

McGuire, R. K., and Cornell, C. A. (1974). ''Live load effects in office buildings.'' *J. Struct. Div.*, ASCE, 100(7), 1351–1366.

Peir, J. C., and Cornell, C. A. (1973). ''Spatial and temporal variability of live loads.'' *J. Struct. Div.*, ASCE, 99(5), 903–922.

Southern Building Code Congress International (SBCCI). (1997). *Standard building code*, Birmingham, Ala.

Tyson, T. (1999). *Parking garage bay layouts*, ASCE-7 Live Loads Task Committee communication, Reston, Va.

Wen, Y. K., and Yeo, G. L. (1999). ''Design live loads for parking garages.'' *Rep., ASCE Struct. Engrg. Inst.*, Reston, Va.