Design Loads for Box Culverts for the SM1600 Design Loading of the Australian Bridge Design Code AS 5100

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ABSTRACT

The new SM1600 design loading of the draft Australian Bridge Design Code AS 5100 is complex. The interpretation of some aspects of the loading and its application in the practical design of large precast reinforced concrete box culverts is not clear. In an attempt to clarify these aspects, published material relating to the development of the SM1600 design loading is reviewed, and the SM1600 loading is briefly compared with previous Australian codes, with overseas codes, and with current knowledge.

It appears that the SM1600 loading was not developed with box culverts and other short span or buried structures in mind. Modifications are suggested to the AS 5100 loading provisions for the design of box culverts, particularly in regard to live load surcharge and compaction pressure. The fatigue design provisions of AS 5100 are such that check for fatigue from moment and shear effects will usually be required for precast box culverts under shallow fill. In the past, this was only required for railway traffic loadings.

The use of the accompanying lane factors, together with the different uniform lane loads associated with the M1600 and S1600 design vehicles, appears to add complexity to the design of large precast box culverts and other short span or buried structures that is not required for these structures.

Accordingly, it is suggested that the SM1600 loading be modified with a view to simplifying it for the purpose of designing culverts and other short span buried structures, to a tandem or a triaxle loading applied without uniform lane load or accompanying lane factors. The limit for the length of short spans and associated axle loads as well as the fatigue loading should be decided as part of this modification.

The adequacy of the current M1600 triaxle load is reviewed using available data from Culway sites, and the use of this data for verification and adjustments of design vehicle loads is briefly discussed.

1 INTRODUCTION AND SCOPE

The 1992 Austroads Bridge Design Code (1) was the first Australian bridge design code issued in limit states format. It was renamed in 1996 as the Australian Bridge Design Code (2) (1996

ABDC). The 1996 ABDC is currently under revision, and is to be issued as Australian Standard AS 5100. A draft of AS 5100.2: Design Loads was issued for public comment on 30/11/ 2000 (3). The SM1600 design loading provisions given in the public comment draft (3) are essentially the same, with the exception of the design lane width, as the SM1600 loading provisions contained in the Vicroads 1999 Technical Note (4) used for some bridge designs to date.

At the time of writing this paper AS 5100 had not been issued. At this time, the SM1600 loading has been little changed from the AS 5100.2 draft for public comment (3) apart from consideration of the effects of HLP 320 or HLP 400 vehicles and the M1600 triaxle group. The fatigue loading provisions have however changed significantly.

The SM1600 design loading is very different from the loading in the 1992 Austroads code and all previous Australian bridge codes, and its interpretation and application in the practical design of reinforced concrete box culverts is not clear in some aspects. An understanding of the basis of the new SM1600 design loading and its code calibration would assist in the interpretation and application of the new SM1600 loading in the practical design of precast reinforced concrete culvert units.

2 DEVELOPMENT OF THE SM1600 DESIGN LOADING

Austroads National Workshops on “Future Directions for Australian Freight Vehicles and Bridge Design Traffic Loading” (5) were held in 1997 to present the “assumptions, information, and logic underpinning the proposed (SM1600) design loading”. The following quotations and observations are made on the 1997 workshop papers:

(a) The “proposed loading was developed to look well beyond recent incremental increases in legal loading to try to find an end point in the evolution of the Australian general freight vehicle”.

(b) The proposed loading model was developed to “simulate the effects of both current and potential future heavy vehicles future traffic”. It appears that this simulation was for the single lane mid-span moments and end span shears for a range of simply supported and continuous bridge spans, and that short spans of less than about 10m received little attention. (see Figure 3 “Approach to the derivation of loading model” in Ref. 5).

(c) The proposed loading model may not simulate the load effects, nor the failure mode for many types of bridge structures. In particular, a precast concrete box culvert structure can have wheel loads applied directly to their top slab whilst their side walls are subject to pressures resulting from the distribution of wheel load effects through adjacent fill. Also, a box culvert is usually buried under fill in which case the live load acting on top of the culvert is a pressure from the distribution of wheel loads through the fill over the culvert. This has load effects that differ from directly applied wheel loads.

(d) There is no mention in the workshop papers of any consideration of the effects of the proposed (SM1600) loading on box culverts and other short span buried structures, or for earth retaining structures such as abutments and walls.

(e) The workshop papers make no mention of any specific details of code calibration of the proposed (SM1600) loading. What is mentioned appears to differ from the '92 Austroads code as follows:

(i) "In the accompanying load approach extreme events are only combined with events that occur frequently. For example, an ultimate limit state vehicle (probability of being exceeded in any one year of 0.005) is not assumed to occur simultaneously with an ultimate limit state wind load but rather an average wind load". The probability of 0.005 (possibly a typing error) is a ten-fold increase of that used in the 1992 Austroads code, which states that an ultimate action has a probability of being exceeded in any one year of 0.0005 (see Table C1.1.9 of Section 1 of '92 Austroads (1)).

(ii) "The load model (SM1600) presented here is likewise developed to represent the average extreme daily event (i.e. the average of the largest events each day)." This is in sharp contrast to the 1996 ABDC, which defines the serviceability limit state load as having a return interval of 20 years. Furthermore, the use of the wording "the average of the largest events each day", implies that the SM1600 serviceability design load is expected to be exceeded a number of times daily by some undefined amount. If this is so then, the SM1600 represents a quantum jump in serviceability requirements compared to previous bridge codes.

(f) It is beyond the scope of this paper to discuss the code calibration of the SM1600 design load in safety terms . Some information relevant to the calibration is provided in references presented below:

(i) The development of the 1992 Austroads code (1) appears to have started in about 1977. It was stated in 1977 at a Seminar on the 1976 NAASRA Bridge Design Specification (6) as follows:- "…it is obvious we must give early consideration to limit state design. The first steps have already been taken by NAASRA in conjunction with the Australian Road Research Board. It is hoped that a draft specification will be available within two years for use with in parallel with our present specification".

(ii) The Australian Road Research Board Internal Report (7) states that:- "The report is a record of notes made for a verbal / visual presentation made in April 1980 at ARRB to a selected group of bridge designers and researchers who are, or are likely to be, involved in the production of an Australian Limit States Bridge Design Code." This report and another by R. A. Dorton and P. F. Csagoly (8) called "The Development of the Ontario Bridge Code", published in October 1977 by the Ontario Ministry of Transport and Communications explain the approach used to develop the Ontario code. It appears that the development methodology of the 1992 Austroads (1) code was based on the methodology used for the 1979 Ontario Bridge Code. It is probable that box culvert structures were excluded from the calibration in developing the 1979 Ontario Code, and probably from the 1992 Austroads (1) code. It should be noted that the Australian national codes for concrete, and for steel building structures were converted to limit states format in the same period. Both were prepared with the stated intention of making them suitable for road and railway bridges (how these code calibrations were extended to cover loadings from road and railway bridges is not known).

(iii) It is probable that the code calibration safety values for precast box culverts (and other buried structures and earth retaining structures) were never calibrated for the ‘92 Austroads (1) code.

3 AS 5100 DESIGN LOADS FOR BOX CULVERTS

Some areas of the proposed AS 5100.2 and the application of the SM1600 loading to box culverts need clarification. These are briefly dealt with below.

3.1 Application of Uniform Lane Load and Accompanying Lane Factors

It appears that the SM1600 design loading was not developed for box culverts. The M1600 moving vehicle has a 6 kN/m uniform lane load but the S1600 stationary vehicle has a 24 kN/m uniform lane load. It is suggested that the application of these differing uniform lane load component, and the accompanying lane factors is inappropriate for precast box culverts, and leads to unnecessary complications in their design and in the design of other buried structures (and earth retaining structures such as abutments and walls).

3.2 Horizontal Earth Pressures Due to Compaction and Live Load.

There is no specific mention of compaction pressure in the proposed AS 5100. This is unfortunate because bridge designers are required to comply with the code, and may consider that compaction pressures may not be of importance in the design of box culverts and other earth retaining structures, such as abutments and retaining walls. The fact is that compaction pressure can be a significant load on such structures.

Some bridge codes have recognised this and specify that compaction pressure be treated as an *additional* horizontal pressure that should be taken into account. The 1994 AASHTO LRFD (9) code states (in Clause 3.11.2) that “the effect of *additional* horizontal earth pressure that may be induced by compaction shall be taken into account”.

Likewise the 1991 Ontario Highway Bridge Design Code (10) states (in Clause 6-7.4.3) that “Compaction Surcharge – For retained fill which is placed and compacted in layers, an *additional* pressure due to compaction of the fill shall be considered. In lieu of detailed calculations, the compaction surcharge given in Figure 6-7.4.3 shall be used.” Figure 6-7.4.3 shows a uniform compaction pressure profile, of 16kPa as a minimum, which is *superimposed* on the horizontal earth pressure due to fill.

The use of the word *additional* implies that compaction pressure should be treated in design as a horizontal earth pressure that is *separate* to the horizontal earth pressure due to fill, and to the horizontal earth pressure due to traffic live loads. The use of the word *additional* may be misleading, particularly for compaction combined with traffic live loads. It is beyond the scope of this paper to deal with this concept in detail, but because of its importance in understanding the application of the SM1600 design loads to box culverts, it is briefly dealt with in Appendix A.

3.3 Horizontal Earth Pressures Due to Live Load.

The interaction of live load induced horizontal earth pressures with horizontal earth pressures due to fill and compaction pressure is also explained in more detail in Appendix A. In brief, the interaction of compaction pressure with traffic live load can in principle be considered to be due to two types of "live load" applied at different stages to the fill. Compaction is one type of "live load", the other type being due to traffic live load. Road traffic live load is analogous to compaction by traffic as "multi-wheel compaction equipment" which is applied to the fill (road) surface *after* construction. Indeed, multi-wheel compaction equipment is frequently used for compaction fill during embankment and road pavement construction.

Traffic live load can be considered as a "second stage" in the fill compaction process. This "second stage" of compaction by traffic live load begins immediately after installation, and continues during the service life of the structure. Accordingly, the horizontal earth pressures due to traffic live load needs to be *superimposed* on, not *added* to, the compaction pressure.

3.4 Vertical Earth Pressure

Earth pressure can be the dominant load on culverts under fill exceeding about 2.5m. Vertical pressure on top slab of a culvert may be computed ordinarily as the weight of fill directly above the culvert. The actual vertical pressure acting on the top of the culvert can vary significantly from the ordinarily computed pressure. The actual pressure on the top of the culvert depends on the soil-structure interaction. A "soil-structure interaction factor" can be calculated for culverts installed in a trench (say "Ft"), and for culverts installed under embankment (say "Fe"). All previous Australian bridge codes provided specific guidance in this area, but Austroads (1) does not. It is suggested that the AASHTO LRFD (9) code (clause 12.11.2.1) provisions for soil-structure interaction factors "Ft" and "Fe" should be included in AS 5100.

4 SERVICEABILITY AND FATIGUE

The fatigue cycles specified in AS 5100.2 draft for public comment (3) are 2,000,000 for the W80 and A160 loads, and 200,000 for the M1600 load. These cycles and magnitude of axle load are likely to generate stresses in reinforcement, stresses of concrete in flexural compression, as well as shear diagonal tension stresses, that are higher than fatigue limits at the serviceability limit state. This is likely to be so for precast box culverts with low fill over their top slab, particularly when Grade 500 MPa reinforcing bars are used as flexural reinforcement at the ultimate limit state.

At time of writing this paper indications are that the fatigue provisions of AS 5100.2 include making the number of fatigue cycles dependant on the road type and the Average Daily Truck Traffic (ADTT). It is understood that for an interstate highway with an ADTT of 1000, the number of fatigue cycles of (0.7 x A160 axle) will be 40 million, and about 10 million of (0.7 x M1600) for a 4m span culvert. This represents a major increase in fatigue loading, and will require the adoption of significantly reduced serviceability stress levels for reinforcement and concrete. Such serviceability stress limits have in the past been necessary only for box culverts under railways. A more accurate and realistic understanding of the live loads, and the live load effects involving the interaction of horizontal earth pressures due to live load and compaction pressure, is required for fatigue analysis of precast box culverts.

5 EXAMINATION OF RTA CULWAY DATA

The final publication draft of AS 5100.2 (11) contains as a load case a design M1600 triaxle group load, with the ultimate load of this triaxle comprising a 36.7 tonne (360 kN) load, a Uniformly Distributed Load component of at least 16.5 kN, a dynamic amplification factor of 1.35 and an ultimate load factor of 1.8, giving a total ultimate triaxle load of 93.3 tonnes (915 kN). Without the dynamic amplification factor, the total ultimate triaxle load is 69.1 tonnes (678 kN).

The publication draft of AS 5100.1 (12), gives the criterion for an ultimate load as that which has a 5% probability of being exceeded during the design life of 100 years. This represents an average return interval of about 2000 years, or a probability of occurrence in any one year of about 0.0005 (this appears to differ from the criteria for an ultimate limit state vehicle in the 1997 workshop papers (5) discussed in detail in Clause 2 above).

To enable a statistical check of the design M1600 triaxle group to be carried out, Culway data for 2001 and 2002 provided by the RTA's Weigh-in-Motion group for Culway sites around NSW comprising the mean, standard deviation and skew (as defined by statistical moments over cube of standard deviation i.e. dimensionless) for all valid measured triaxles in those years was obtained.

Culway measured loads are reported as static axle group loads. Each site is calibrated using a calibration truck(s), and a dynamic factor is used to convert the measured dynamic loads to static vehicle loads. Hence, for comparison purposes, the dynamic effects should be discounted.

Given the above, a useful statistical check from Culway data of the adequacy of the design M1600 triaxle load would comprise:

> "For the mean and standard deviation of measured triaxle loads from Culway data, the ultimate triaxle load without dynamic effects of 69.1 tonnes would currently be adequate

> if in all cases the mean plus 3.283 times the standard deviation of those measured loads is less than 69.1 tonnes, for a year's data; and the mean plus 3.474 times the standard deviation for six months of data.”

The maximum value for “Mean + 3.5 x Standard Deviations” for the data from selected NSW Culway sites was 44.9 tonnes, which is much less than the 69.1 tonnes ultimate M1600 triaxle load described above.

However, the RTA’s Culway data for 2001 and 2002 detected some isolated extreme loads on triaxle groups of greater than 60 tonnes, but none more than 70 tonnes, which indicates that the statistical test above does not tell the whole story, and that some grossly overloaded vehicles are present at times on the road network.

It should be noted that only data from sites for which some confidence exists was selected for this brief study. The RTA’s Weigh-in-Motion leader expressed the opinion that the data could be too unreliable for design purposes, and that RTA Culway data for 2001 and 2002 should not be relied upon in deciding live loads. To allow use of Culway data in the future for arriving at or verifying design vehicles for bridge design, as much useful and essential data about vehicle loads can be obtained from this source, it is recommended that the existing RTA Culway sites and those in other states be repaired and calibrated, and new sites placed on representative routes to enable reliable vehicle load data to be obtained. The placement of Culway instrumentation is not expensive, and the data can potentially be of high quality.

As it appears that triaxle loads up to or in excess of the ultimate 69.1 tonnes have not been recorded in current data, and these loads are not likely to have yet occurred, because no loads greater than 70 tonnes have been measured in the 2001 and 2002 RTA Culway data, this gives some reassurance that the proposed M1600 triaxle load in the final publication draft of AS 5100.2 (11) is satisfactory for current loads, notwithstanding doubts about the reliability of the Culway data.

The adequacy of the M1600 triaxle load for the design of culverts and other short span structures should be verified and adjusted if necessary over the next few years after more reliable data is collected from Culway sites across Australia. The proposed M1600 triaxle load should be sufficiently conservative for current triaxle loads, and may have some margin for future load growth. Whether this margin is adequate for the projected growth in vehicle mass over the next 100 years is yet to be ascertained.

6 SUMMARY AND CONCLUSIONS

(a) It appears that the SM1600 design loading was not developed for box culverts. The M1600 moving vehicle has a 6 kN/m uniform lane load but the S1600 stationary vehicle has a 24 kN/m uniform lane load. It is suggested that the application of these differing uniform lane load component, and the accompanying lane factors is inappropriate for precast box culverts, and leads to unnecessary complications in their design.

(b) It is suggested that the SM1600 loading should be reviewed with a view to simplifying it for the purpose of designing culverts and other short span buried structures, to a tandem or a triaxle loading applied without a uniform lane load or accompanying lane factors. This review should include examination of Culway data and records of measured overloaded vehicles and axles and any in service performance data of culverts and related structures, to define design loads based on an acceptable level of risk of failure. The axle loads and the fatigue loading should be decided as part of this review.

To simplify application in design, and pending such a review, it is suggested that the Accompanying Lane Factor should be 1.0 for all lanes when the SM1600 loading is applied to box culverts and similar structures.

(c) Compaction pressure should be added as a specific load in AS 5100, and its interaction with live load induced horizontal earth pressure clarified. Live load surcharge will need to be redefined in this process.

For design and pending review, it is suggested that compaction pressure be allowed for as recommended in Section 3.2 above and Appendix A of this paper.

(d) For buried box culverts, AS5100 should include soil-structure interaction factors for the calculation of the vertical pressure acting on the top slab of a culvert. Pending review, it is suggested that the AASHTO LRFD (9) code (clause 12.11.2.1) provisions for soil-structure interaction factors “Ft” and ”Fe” should be used in design.

(e) The proposed M1600 triaxle load appears to be sufficiently conservative for current triaxle loads, with some margin for future load growth. The adequacy of the load and whether adjustments should be made should be ascertained from the collection of reliable Culway data on representative routes across Australia.

7 ACKNOWLEDGMENT

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8 DISCLAIMER

The opinions expressed in this paper are entirely those of the authors, and do not necessarily represent the policy of the RTA.

APPENDIX A

A1 HORIZONTAL EARTH PRESSURE DUE TO COMPACTION AND LIVE LOAD.

A1.1 Fill placed against retaining structures is usually compacted in (thin) layers by mechanical compaction equipment which applies high vertical pressure at the surface of each layer. A theory of compaction for free draining cohesionless fill placed against a rigid wall was published by Prof. B Broms (13) proposed that, at any depth in the fill, the horizontal earth pressure increment is equal to the K_o (at rest) coefficient times the vertical pressure increment. Thus at any depth of fill (say D_f), the compaction added pressure (say C_{ad}) increment is added to the existing horizontal pressure due to fill (say F_{hp}) to calculate the compacted fill pressure (say C_{fp}). For fill density = γ kN/m3, D = depth of fill (m), the compacted fill pressure (C_{fp}) can be expressed as:

Compacted fill pressure = Fill horizontal pressure + Compaction added pressure.

$$C_{fp} = F_{hp} + C_{ad}$$

$$C_{fp} = K_o \gamma D + C_{ad}$$

A1.2 Broms (13) shows that the compacted fill pressure (C_{fp}) remains in the fill after removal of the compactor, except that in his words, “if the distance below the ground surface is less than the critical distance Z_{cr}, the lateral earth pressure against the wall will decrease when the compactor is removed”. The compacted fill pressure (C_{fp}) is thus a maximum at depth Z_{cr}, and reduces linearly to zero at the surface of the fill. (This must be the case at the end of compaction of each layer of fill, since the vertical pressure at the surface of the fill is returns to zero when the compactor is removed).

A1.3 At depths of fill greater than the critical depth Z_{cr}, the horizontal compacted fill pressure profile (C_{fp}) consists of a series of peaks of value (C_{fp}), with each peak associated with a compaction layer. The (C_{fp}) pressure peaks can be joined with a vertical line and the compacted fill pressure profile thus approximated by a uniform (vertical line) value of C_{fp} for depths greater than Z_{cr}.

A1.4 As additional layers are compacted, a depth of fill (say D_{cf}) is reached where the compacted fill pressure is exceeded by the fill horizontal pressure i.e., $K_o\ \gamma\ D_{cf} > C_{fp}$.

A1.5 Figure A1.5 depicts the compaction pressure superimposed on the earth pressure.

A1.6 The compacted fill pressure profile can thus be approximated in two parts. The first part starts with zero value at the fill surface, and then increases linearly to a maximum value of C_{fp} at a depth of Z_{cr} (typically about 0.6m). The second part remains constant at C_{fp} from Z_{cr} to a depth of D_{cf} (typically 2m - 2.5m) where it is exceeded by the fill horizontal pressure alone. For the first part of the pressure profile it is generally conservative to assume that maximum compaction pressure value C_{fp} extends from Z_{cr} to the surface of the (road) fill. (This approximation is made in clause 6-7.4.3 of the 1991 Ontario Highway Bridge Design Code (10)).

It then remains constant at the maximum C_{fp} value down to some depth of fill D_{cf} where the horizontal earth pressure due to fill exceeds C_{fp}.

At depths of fill greater than D_{cf}, the uniform C_{fp} value profile is superseded (replaced) by the higher values of the horizontal earth pressure profile due to fill.

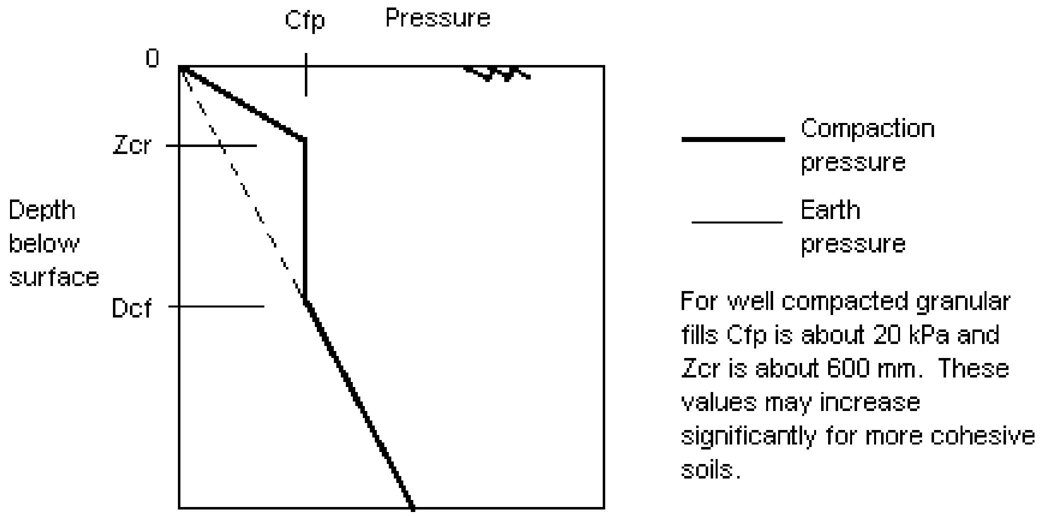

Figure A1.5: Compaction pressure superimposed on earth pressure

A1.7 Accordingly, the compaction horizontal earth pressure profile is *superimposed* on the horizontal earth pressure due to fill for depths of fill less than D_{cf}, and it has no effect at depths of fill greater than D_{cf}. Thus the compaction induced horizontal earth pressure is not an *additional* horizontal earth pressure. In design, compaction pressure should be considered as a horizontal earth pressure that is *superimposed* on, not *added* to, the horizontal earth pressure due to fill.

A1.8 Experimental work, including full scale compaction testing by Carder, Murray and Krawczyk (14) and by Symons and Murray (15) have confirmed the development of residual horizontal pressures due to compaction for sands, silty clays and clays, and support the use of the coefficient of earth pressure at rest K_o as recommended by Broms. The work at the TRRL also confirms that horizontal earth pressure due to compaction:-

(1) remains as a permanent pressure for granular non cohesive soils.

(2) reduces with time to the K_o value for silty clays and clays.

Carder, Murray and Krawczyk (14) and Symons and Murray (15) reported that four months after compaction using silty clay, the compaction pressures had reduced to lie close to the K_o line. The reduction for heavy clay was about 12% over a four week period.

The above experimental findings have significant ramifications in design for load combinations of compaction pressure with horizontal earth pressure due to fill, and with live load induced horizontal earth pressures. This is explained below.

A2 HORIZONTAL EARTH PRESSURE DUE TO LIVE LOAD.

The ramifications in design for load combinations of horizontal earth pressure due to fill with compaction and with live load induced horizontal earth pressures is explained below.

A2.1 Nechvoglod (16) extends the compaction theory outlined by Broms in 1971 to explain the nature of live load induced horizontal earth pressure, and its interaction with the horizontal earth pressures due to fill, and with the horizontal earth pressure due to compaction. Nechvoglod suggests that the horizontal earth pressures produced by traffic live load (wheel loads) is produced in an analogous manner to compaction. The vertical earth pressure increments due to traffic live load can be calculated from the Boussinesq vertical stress equation (as for compaction), and the corresponding horizontal earth pressure increase on a rigid wall calculated as being equal to K_o times the vertical pressure increase (as for compaction).

A2.2 When considering the interaction of compaction with traffic live load, the increments in horizontal earth pressure can in principle be considered to be due to two types of "live load" applied at different stages to the fill. Compaction is one type of "live load", and the other type is due to traffic live load. Road traffic live load is analogous to "multi-wheel compaction equipment" which is applied to the fill (road) surface *after* construction. Indeed, multi-wheel compaction equipment is frequently used for compaction fill during embankment and road pavement construction. Accordingly, traffic live load can be considered as a "second stage" in the fill compaction process. This "second stage" of compaction by traffic live load begins immediately after installation, and continues during the service life of the structure. Accordingly, the horizontal earth pressure due to traffic live load needs to be *superimposed* on, not *added* to, the compaction pressure. Compaction has no effect at depths of fill greater than Dcf (as explained for the case of compaction alone). However, traffic live loads may produce some *additional* horizontal pressures at depths greater than Dcf, if the traffic wheel loads produce additional vertical pressure at this depth.

A2.3 The above explanation of live load induced horizontal pressure as being analogous to compaction pressure clearly differs in concept and form from live load "Surcharge Loads" as specified in AS 5100 and in Clause 2.11.3 "Surcharge Loads" of the 1992 Austroads (1) code.

A2.4 No simple comparison is possible between the SM1600 loading treated as a "multi-wheel compaction equipment" and this AS 5100 "Surcharge Loads". For any such a comparison, it is meaningless to include the uniform lane load component of 6kN/m with the M1600 vehicle and the 24kN/m for the SM1600 vehicle, since the SM1600 loading was clearly not developed for box culverts (or for other buried structures and retaining walls). In any event, any such comparison should be on the understanding that compaction pressure can only exist at a "rigid" wall or surface. In contrast, the code "Surcharge Loads" only come into effect when the wall or surface "yields" under the action of a soil "failure" wedge, with the live loads located and acting on this "failure" wedge. Limit State load combinations should take this into account.

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